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وزارة الموارد المائية والرى

مكتب رئيس قطاع الخزانات والقناطر الكبرى

الدور ال ١٩ بمبنى الوزارة – كورنيش النيل – إمبابة

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السيد المهندس / رئيس الإدارة المركزية-المهندس المقيم

لمشروع إنشاء قناطر نجع حمادى الجديدة ومحطتها الكهرومائية

تحية طيبة و بعد ٠٠٠

نتشــرف بــأن نرســل لسيادتكم نســخة مــن تقريــر التصــميم – الجــزء الأول للأعمــال المدنيــة

المعد بمعرفة الاستشارى "لاماير" أثناء فترة Tender Design وذلك للإحاطة.

وتفضلوا بقبول فائق الاحترام ٠٠٠

رئيس قطاع الخزانات و القناطر الكبرى

"أ.د. /محمد بهاء الدين أحمد "

٢٠٢٤/٢/١٧

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ARAB REPUBLIC OF EGYPT

# New Naga Hammadi Barrage and Hydropower Plant

## PROJECT IMPLEMENTATION UNIT

Ministry of Water Resources and Irrigation
Reservoirs and Grand Barrages Sector
&
Ministry of Electricity and Energy
Hydro-Power Plants Executive Authority

## TENDER DESIGN REPORT

# VOLUME 1
# - MAIN CIVIL WORKS -

SEPTEMBER 2000

**NAGA HAMMADI BARRAGE DEVELOPMENT CONSULTANTS**
Lahmeyer International - Electrowatt - Sogreah

in Association with

Arab Consulting Engineers                    Electric Power Systems Eng. Co.

# Tender Design Report

## VOLUME 1 - MAIN CIVIL WORKS

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# VOLUME 1
# MAIN CIVIL WORKS

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# LIST OF ABBREVIATIONS

| | |
|---|---|
| A | Ampere |
| AC | Alternate current |
| ACI | American Concrete Institute |
| AISI | American Iron and Steel Institute |
| ANSI | American National Standards Institute |
| A.R.E. | Arab Republic of Egypt |
| ASHRAE | American Society of Heat, Refrigerating and Air Conditioning Engineers |
| ASME | American Society of Mechanical Engineers |
| ASTM | American Society for Testing Materials |
| AVR | Automatic Voltage Regulator |
| AWS | American Welding Society |
| A/C | Air conditioning |
| $B'_{lim}$ | Stability coefficient |
| BH | Borehole |
| BS | British Standards |
| BSI | British Standards |
| BW | Borehole |
| B&W | Black & white |
| $C°$ | Celsius (Temperature), degrees |
| CB | Control Board |
| CCD | Charge Couple Device |
| CCITT | International Telegraph and Telephone Consultative Committee |
| CIGRE | International Conference on Large High-Voltage Electric Systems |
| cm | Centimeter |
| CT | Current Transformer |
| d | Thickness of structural member |
| daN | Deca Newton = 10 N |
| DC | Double Current |
| DDC | Digital Direct Controlling System |
| DIN | Deutsche Industrie Norm (German Industrial Standards) |
| DX | Direct expansion type of fan coil units |
| $D_x$ | Grainsize at percentage finer x, in mn |

| d/s | Downstream |
| $D_{50}$ | Diameter of 50 % finer |
| EAB | Empfehlungen des Arbeitskreises "Baugruben" (recommendations of committee for "construction pits" |
| EBPH | Emergency Board Powerhouse |
| EBSN | Emergency Board Sluiceway and Navigation Lock |
| EC | Egyptian Code |
| EC2 | Euro Code 2 |
| EEA | Egyptian Electricity Authority |
| EG | Environmental Group |
| EGSA | Egyptian General Survey Authority |
| EIA | Electronics Industries Association |
| EIA | Environmental Impact Assessment |
| EN | Euro Norm |
| EPS | Electric Powers System's Company |
| FoS | Factor of Safety |
| g | Gravity Acceleration = 9.81 m/s$^2$ |
| g | Gram |
| GANT | General Authority of Nile Transport |
| GPS | Global Positioning System |
| GWh | Giga Watt hours |
| H | Net head |
| h | hour(s) |
| HAD | High Aswan Dam |
| HEC | Hydraulic Engineers Corps |
| HFW | High Frequency Welding |
| HMW | Hydro-Mechanical Works |
| HP | High pressure |
| Hpa | Hecto Pascal |
| HPP | Hydropower Plant |
| HPPEA | Hydro-Power Plants Executive Authority |
| HRI | Hydraulic Research Institute |
| HV | High Voltage |
| HVAC | Heating, ventilation and air-conditioning |
| Hz | Hertz |
| I | Hydraulic gradient (h/l) |

| IEC | International Electronical Commission |
| IEEE | Institution of Electrical and Electronic Engineers |
| IPCEA | Insulated Power Cable Engineers Association |
| ISO | International Organisation for Standardisation, Switzerland |
| ITU-T | International Telecommunication Union - Telecommunication Standardisation Sector |
| JEC | Standard of the Japanese Electrical Technical Committee |
| JIS | Japanese Industrial Standard |
| K | Kelvin |
| $k_i$ | Earth pressure |
| $k_h$, $k_v$ | Permeability, horizontal and vertical |
| kA | Kilo Ampere |
| kg | Kilogram |
| km | Kilometer |
| kN | Kilo Newton = 1,000 Newton |
| kV | Kilo Volt |
| kVA | Kilo Volt Ampere |
| kW | Kilo Watt |
| lux | Illumination level or unit |
| LV | Low Voltage |
| m | Meter |
| $m^2$ | Square meter |
| $m^3$ | Cubic meter |
| m asl | Meter above sea level - elevation |
| max | Maximum |
| MCB | Miniature Circuit Breaker |
| MCC | Motor Control Center |
| MCE | Maximum Credible Earthquake |
| MC90 | Model Code 1990 |
| MEE | Ministry of Electricity and Energy |
| MHZ | Megahertz |
| mg/l | Milligram per liter |
| min | Minimum |
| min | Minute |
| mio | Million = 1,000,000 |
| ml | Milliliter |

| mm | Millimeter |
| MMI | Maximum Magnitude Intensity |
| MN | Mega Newton |
| MPa | Mega Pascal |
| MV | Medium Voltage |
| MVA | Mega Voltampere |
| MW | Mega Watt |
| MWRI | Ministry of Water Resources and Irrigation |
| $m^3/s$ | Cubic meter per second - discharge |
| N | Newton |
| NE | Northeast |
| NEMA | National Manufacturers Association |
| NF | Norm Francaise (French standards) |
| NFPA | National Fire Fighting Protection Association |
| NHB | Naga Hammadi Barrage |
| NL | Navigation Lock |
| Nm | Newton meter |
| no(s). | Number(s) |
| NOL | Normal Operating Level |
| NW | Northwest |
| OHTL | Overhead Transmission Line |
| ONAF | Oil natural air forced |
| ONAN | Oil natural air natural |
| OPGW | Optical Fiber Ground Wire |
| P | Pressure |
| PABX | Private Automatic Branch Exchange |
| PH | Powerhouse |
| pH | Value indicating acidity/alkalinity of fluid |
| PID | Proportional Integrated Digital Regulator |
| PIU | Project Implementation Unit (MWRI and MEE/HPPEA) |
| PLC | Programmable Logic Controller |
| ppm | Parts per million |
| PVC | Polyvinyl-chloride |
| Q | Discharge |
| rms | Route mean square |

| | |
|---|---|
| rpm | Revolution per minute |
| $S_n$ | Rated power |
| s | Second |
| SAP90 | Software programme |
| SCADA | Supervisory, Control and Data Acquisition |
| SDV | Standard deviation |
| sec | Second |
| SF6 | hexagen fluid gas insulated encapsulated switchyard |
| SIS | Sveriges Standdardiserings Kommission (swedish standards) |
| SMAW | Shielded Metal Arc Welding |
| SMB | Sverdrup- Munk-Bretschneider method |
| SPD | Standard Proctor Density |
| SPT | Standard Penetration Test |
| SW | Southwest |
| t | Ton = 1,000 kg |
| T | Temperature |
| TDS | Total Dissolved Solids |
| TIG | Tungsten Inert Gas Welding |
| TV | Television |
| U | Uniformity coefficient $D_{60}/D_{10}$ |
| UCB | Unit control board |
| UCS | Uniaxial Compressive Strength |
| UK | United Kingdom of Great Britain |
| $U_n$ | Rated Voltage |
| UPS | Uninterrupted Power Supply |
| UPS | Unified Power System |
| USA | United States of America |
| USACE | United States Army Corps of Engineers |
| USBR | United States Bureau of Reclamation |
| UTS | Ultimate Tensile Strength |
| u/s | Upstream |
| V | Volt |
| v | velocity |
| VA | Volt Ampere |
| VDE | German Society of Electrical Engineers |

| VDI | Verein Deutscher Ingenieure (Society of German Engineers) |
| W | Watt |
| WHCO | World Health Conference Rio de Janeiro 2000 |
| WL | Water Level in m asl |
| WM | Water Management |
| $WR^2$ | Moment of inertia |
| $W_X$ | Weight of the X % size in the gradation of riprap |
| $W_{50}$ | Weight of the 50 % size in the gradation of riprap |
| XLPE | Cross-linked Polyethylene |
| YS | Yield Strength |
| $\gamma$ | Unit weight |
| $\Omega$ | Ohm, electric resistance |
| $\rho$ | Specific weight, in kg/m$^3$ |
| $\sigma$ | Thoma coefficient |
| $\varnothing$ | Diameter |

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# CHAPTER 1

# PREAMBLE

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## 1.    PREAMBLE

The layout of the New Barrage structures, with its axis some 3.5 river kilometres downstream of the existing barrage, was selected during studies lasting from 1993 to 1997, which included a substantial amount of geotechnical investigation at the site and over the area between the existing and the New Barrage.

The purpose of the New Barrage is:

- to maintain a constant headpond level of 65.90 m asl, which is considered to be adequate to control the irrigation releases through the left bank and right bank head regulators which feed the main irrigation canal on both sides of the Nile, down to the area of Assiut,

- to enable sluicing for the increasing fleet of ships operating on the Nile, by a two chamber lock with the new standard dimensions set by the GANT,

- to safely evacuate the emergency release from the HAD of 7,000 $m^3$/s through the gated sluiceway,

- to generate power by run-of-river operation with the releases to the downstream riverbed,

- to enable public road traffic across the river by a road bridge on the barrage, complying with one lane road standards.

Together with the implementation of the New Barrage and impoundment of the intermediate river reach, the gates of the old barrage and its two navigation locks will be permanently opened. Shipping will be through the gaps remaining from of the old barrage locks. The old barrage, then no longer exposed to a differential head, will continue to be used as a bridge for local traffic.

Both old head regulators, west and east, located some hundred metres upstream from the old barrage, will be rehabilitated and upgraded with their equipment to control the irrigation canal water levels in accordance with the higher headpond and the irrigation demands.

The design description of the main civil works together with the applied criteria, load assumptions, and design methods form Volume 1 of the Tender Design Report.

The Tender Design Report is continued by Volume 2 describing the criteria, design methods applied and requirements on the equipment, regarding the "Hydromechanical Equipment" (Hydraulic Steel Structures), the "Bulb Turbine and Generators", and the "Electrical Equipment" of the powerhouse and barrage structures, including the switchyard. For the transmission line, which is still under study by the Electric Power System's Company (EPS), this design report is restricted to a summary of the main design criteria.

The planning criteria which led to the layout of the barrage structures are given in Chapter 2, followed by a conceptual description of the layout and its main components in Chapter 3, and the detailed design criteria and calculations in Chapters 4 to 13.

Many of the design constraints and criteria result from the geotechnical investigations described in Volume 4, and from the hydraulic model investigations with the sluiceway detail model, the barrage model and the navigation lock model described in separate reports available at the MWRI.

Volume 3 of the "Tender Design Report" is formed by a reduced copy of the Tender Drawings, which also form part of the Tender Documents.

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# CHAPTER 2

# RIVER AND CLIMATIC CONDITIONS

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## 2.    RIVER AND CLIMATIC CONDITIONS

### 2.1    River Flow at Naga Hammadi Barrage

River flow values are given after extraction of discharges by the eastern and western irrigation canals (head regulator intakes). The river flow is the result of releases from Lake Nasser and the sum of the abstraction upstream of the existing barrage. Normally there is no inflow into the river downstream of Lake Nasser. River flow rates are adjusted according to an irrigation demand schedule in intervals of 10 days.

### Table 2.1: Average Flows at Naga Hammadi Barrage

| Month | Average m$^3$/s | Maximum 10-Day m$^3$/s | Minimum 10-Day m$^3$/s |
|---|---|---|---|
| January | 638 | 815 | 350 |
| February | 1,033 | 1,070 | 963 |
| March | 1,338 | 1,465 | 1,160 |
| April | 1,347 | 1,381 | 1,326 |
| May | 1,569 | 1,796 | 1,372 |
| June | 2,292 | 2,320 | 2,252 |
| July | 2,203 | 2,223 | 2,172 |
| August | 1,991 | 2,063 | 1,892 |
| September | 1,412 | 1,669 | 1,119 |
| October | 1,020 | 1,059 | 997 |
| November | 1,082 | 1,231 | 910 |
| December | 679 | 694 | 654 |
| Annual | 1,380 | - | - |
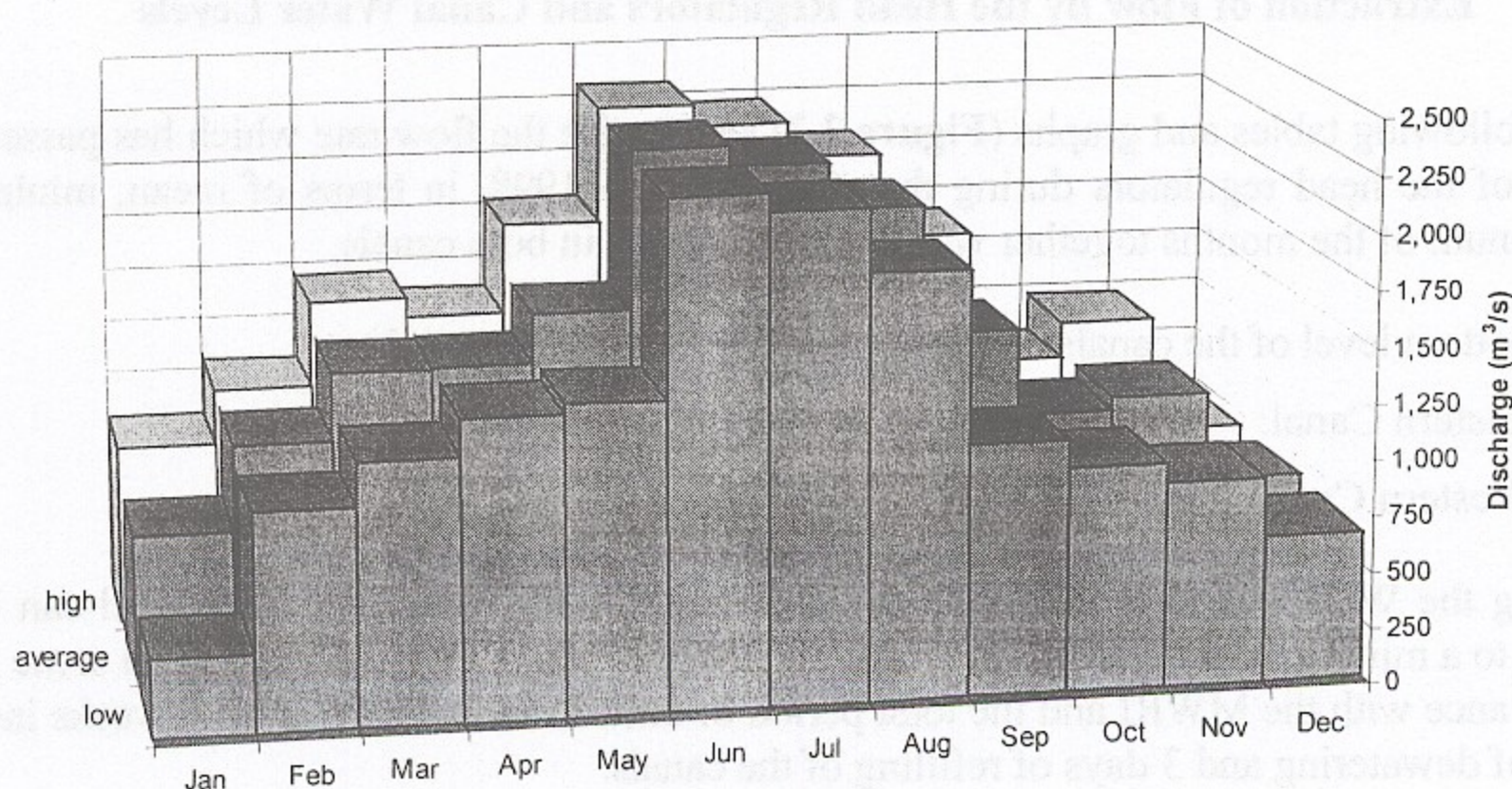


### Figure 2.1: Flows at Naga Hammadi Barrage

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Daily discharges during the summer months June to August peak at around 2,400 m³/s. The flow reduces to as low as 350 to 400 m³/s for short periods in December and January. This period, referred to as winter closure, marks the period when releases are reduced significantly from the HAD to allow maintenance of the irrigation systems throughout Egypt. This is undertaken in mid- to late-December in Upper Egypt and early to mid-January in Lower Egypt.

The flow duration curve based on 10 days of release intervals is presented in **Figure 2.2**.
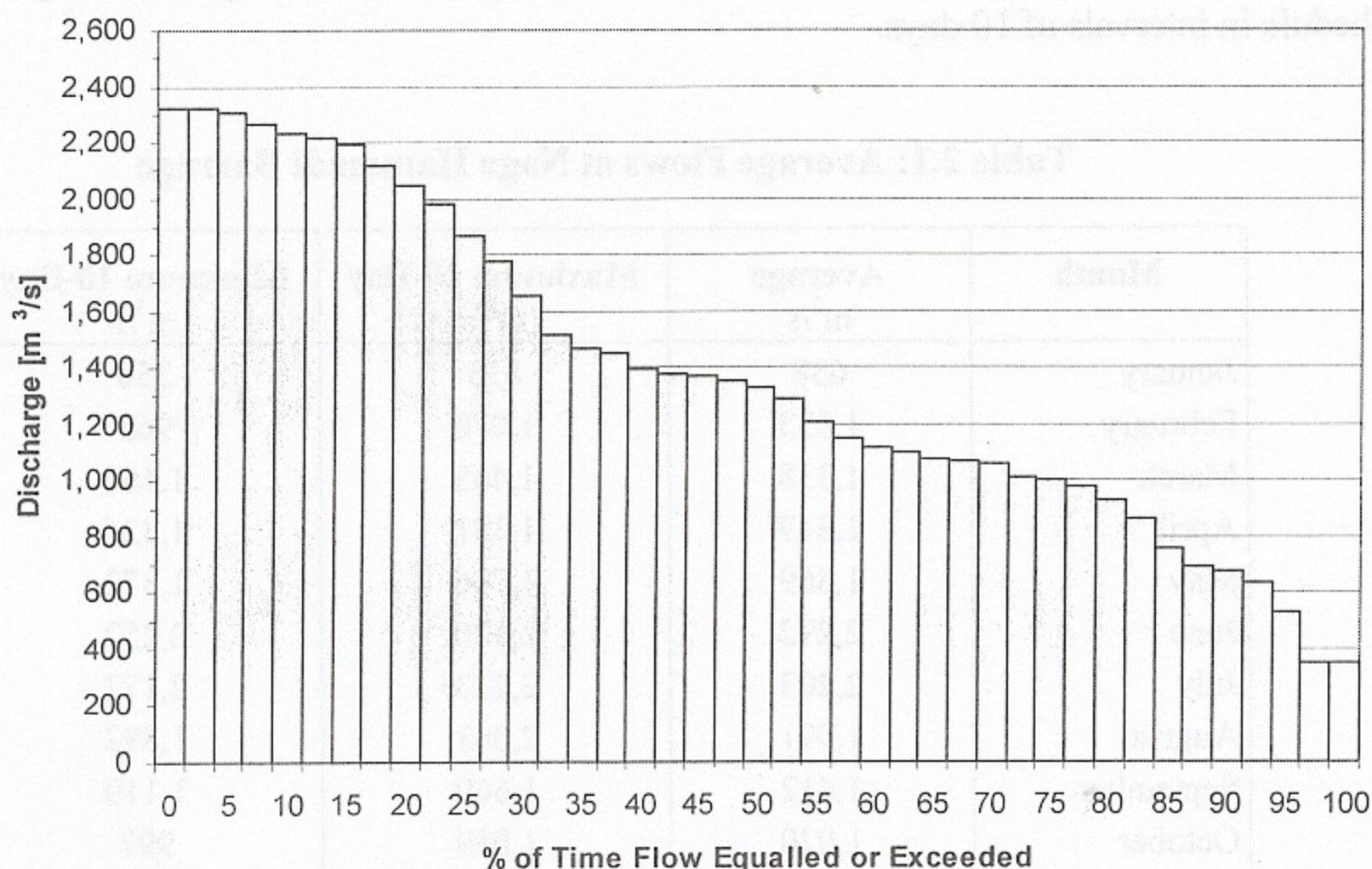


**Figure 2.2: Flow Duration Curve at Naga Hammadi Barrage**

## 2.2 Extraction of Flow by the Head Regulators and Canal Water Levels

The following tables and graphs (**Figure 2.3**) summarise the flow rate which has passed through each of the head regulators during the years 1990 to 1998, in terms of mean, minimum, and maximum of the months together with the water levels in both canals.

The bottom level of the canals downstream of the head regulators is at:

- Eastern Canal: 61.25 m asl,

- Western Canal: 60.50 m asl.

During the Winter Closure Period of the irrigation system, the canal water level can be drawn down to a minimum of about 0.5 m above the bottom level. This has to be agreed some 2 months in advance with the MWRI and the total period of drawdown could be up to 6 weeks including 3 days of dewatering and 3 days of refilling of the canals.

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| Eastern (Foudia) Canal - Discharge m³/s 1990 - 1998 | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | mean |
| mean | 36.8 | 38.8 | 38.2 | 34.1 | 37.3 | 42.3 | 44.3 | 42.9 | 40.9 | 34.4 | 33.9 | 33.5 | 38.1 |
| min | 30.1 | 31.8 | 28.9 | 26.7 | 21.4 | 23.1 | 23.1 | 23.1 | 27.8 | 23.1 | 23.1 | 23.1 | 29.7 |
| max | 40.5 | 46.3 | 48.6 | 42.2 | 44.6 | 55.8 | 55.8 | 53.8 | 52.1 | 45.7 | 41.7 | 45.1 | 43.3 |



| Eastern (Foudia) Canal - Downstream Waterlevel (m.a.s.l.) 1990 - 1998 | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | mean |
| mean | 64.92 | 64.94 | 65.06 | 65.12 | 65.16 | 65.23 | 65.27 | 65.22 | 65.13 | 64.90 | 64.85 | 64.80 | 65.05 |
| min | 64.75 | 64.70 | 64.85 | 64.98 | 65.00 | 64.90 | 64.90 | 64.90 | 64.70 | 64.70 | 64.60 | 64.20 | 64.89 |
| max | 65.03 | 65.03 | 65.30 | 65.30 | 65.33 | 65.70 | 65.70 | 65.55 | 65.40 | 65.03 | 64.97 | 65.03 | 65.20 |



| Western (Faroukia) Canal - Discharge m³/s - 1990 - 1998 | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | mean |
| mean | 126.2 | 110.7 | 91.4 | 93.9 | 109.7 | 134.3 | 134.0 | 125.8 | 117.3 | 91.8 | 101.9 | 116.1 | 112.8 |
| min | 118.1 | 81.0 | 64.8 | 79.9 | 94.9 | 118.1 | 119.2 | 113.4 | 110.0 | 76.4 | 83.3 | 74.1 | 103.4 |
| max | 143.5 | 164.4 | 115.3 | 115.7 | 125.7 | 151.6 | 151.6 | 143.5 | 143.5 | 110.0 | 136.3 | 150.5 | 127.1 |



| Western (Faroukia) Canal - Downstream Waterlevel (m.a.s.l.) - 1990 - 1998 | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | mean |
| mean | 64.73 | 64.36 | 64.03 | 64.16 | 64.44 | 64.89 | 64.89 | 64.77 | 64.63 | 64.17 | 64.31 | 64.46 | 64.49 |
| min | 64.60 | 63.80 | 63.90 | 64.00 | 64.10 | 64.60 | 64.63 | 64.55 | 64.50 | 64.05 | 64.15 | 63.75 | 64.40 |
| max | 64.80 | 65.00 | 64.30 | 64.70 | 64.80 | 65.10 | 65.20 | 65.00 | 64.90 | 64.40 | 64.70 | 64.90 | 64.70 |
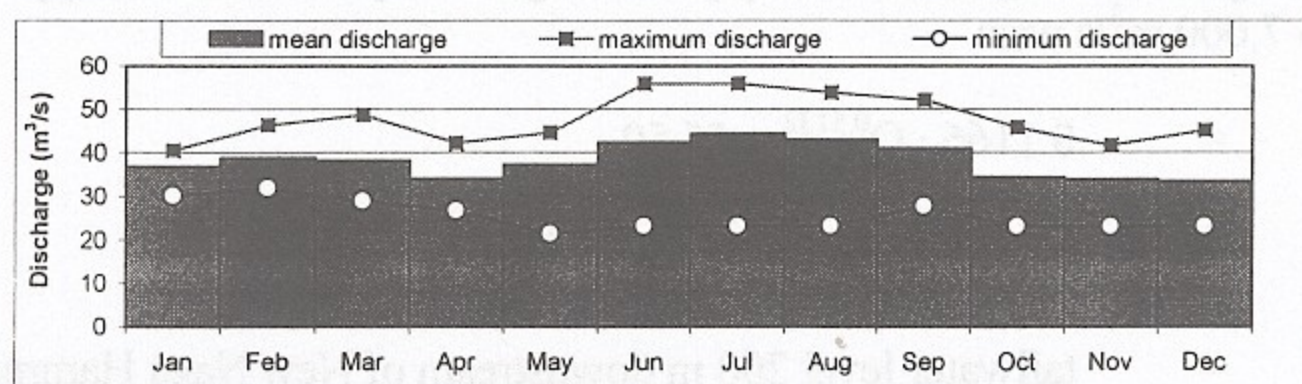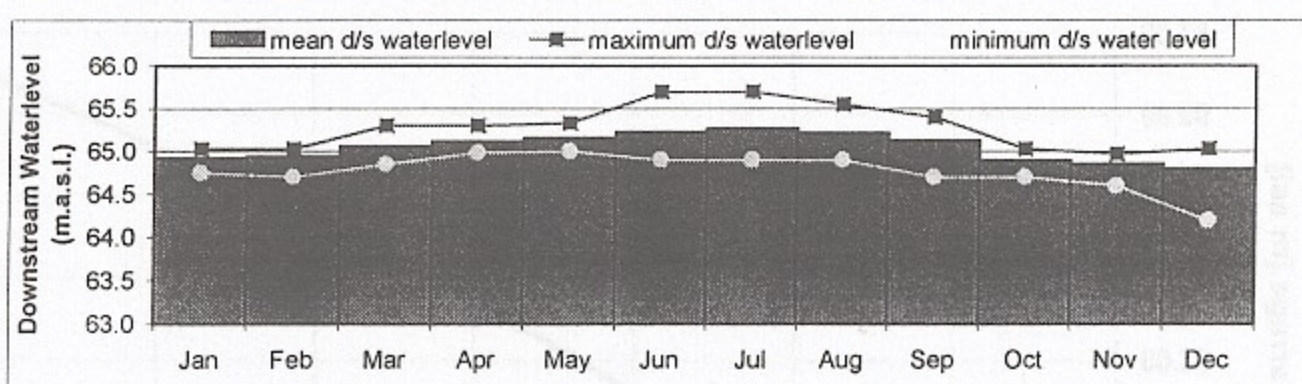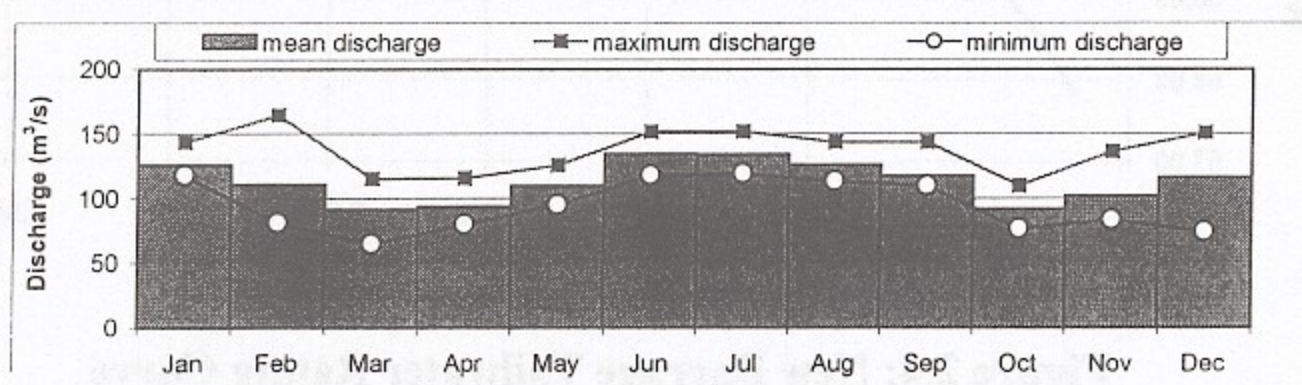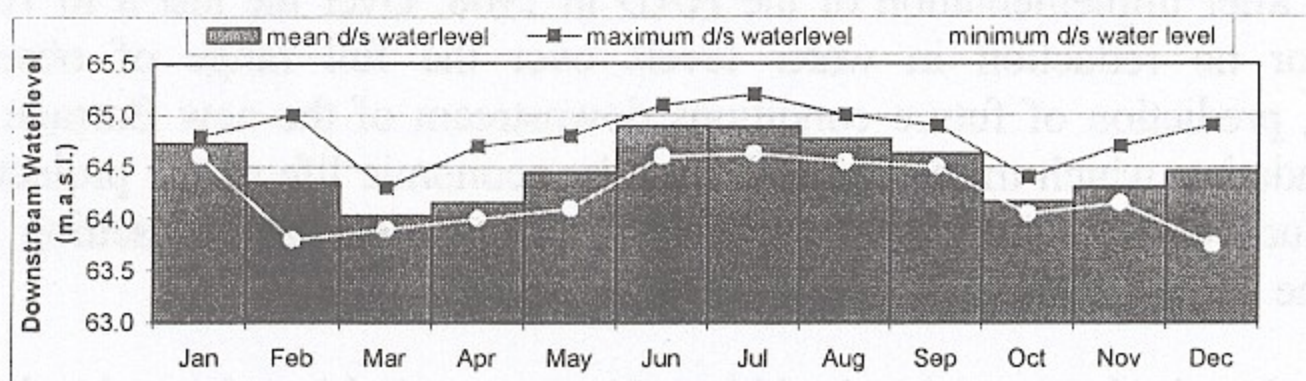


**Figure 2.3: Flow and Water Level at Eastern and Western Head Regulators**

## 2.3    Rating Curves of River Levels under Natural Conditions

The stage discharge relationship at the New Barrage location was calculated by water level measurements in the lower range and extrapolation by water surface calculations to the higher range. The stage - discharge relationship prevailing in the year 1996 and applicable to flow rates from 350 to 7,000 m³/s was:

$$WL \quad = \quad 0.1166 \cdot Q^{0.5132} + 55.50$$

or

$$Q \quad = \quad 65.8555 \cdot (WL - 55.50)^{1.94856}$$

where

WL    =    tailwater level 200 m downstream of New Naga Hammadi Barrage, m asl
Q     =    discharge 200 m downstream of the axis of the new Barrage, m³/s.

This curve is shown on **Figure 2.4**.
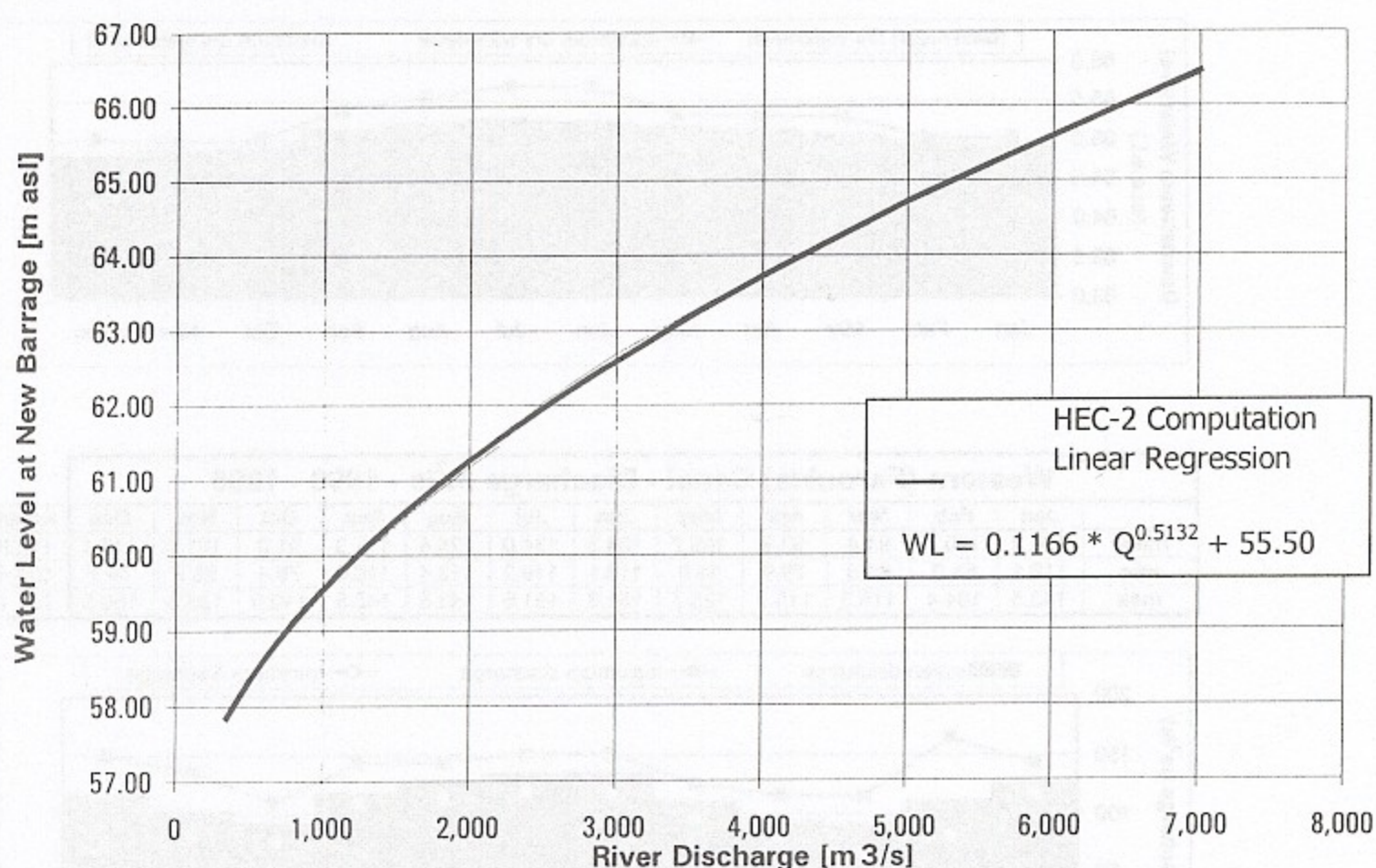


**Figure 2.4: New Barrage Tailwater Rating Curve**

Hydrographic surveys along the River Nile have indicated moderate changes in bed levels, and results presented by the MWRI (1991) suggest the majority of this reduction occurred only immediately after implementation of the HAD in 1968. Over the last 8 to 10 years, there has been little or no reduction in water levels over the full range of observed discharges. Nonetheless, prediction of future conditions downstream of the new Barrage, particularly any further degradation which might occur during the economic life of the project, was considered necessary. Such changes are of consequence for navigation, turbine setting height, structural stability of the barrage components, and generator dimensioning.

The possible drop in the water level within a 50 years period is estimated to be some 0.7 to 0.8 m. The estimated long-term tailwater rating curve results in:

$$WL \quad = \quad 0.0879 \cdot Q^{0.5463} + 54.90$$

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An intermediate rating curve for the period after thirty years of operation has been established as follows:

$$WL = 0.0818 \cdot Q^{0.55801} + 55.20$$

The variables are defined above. This relationship was applied in the design of the project to the turbine setting height, generator dimensioning, river bed and navigation lock sill levels, and sluiceway apron levels.

## 2.4 Climatic Conditions at Site

Egypt's climate except for the winter months of November, December, January and February is hot and dry. Temperatures increase from the north to the south. Average temperatures range from 20° C on the Mediterranean Coast to 26° C in Aswan, maximum temperatures for the same place can get up to 31° C and 50° C respectively. At night in winter the temperatures sometimes drop as low as 8° C even in the south. In winter and even well into spring it can get chilly in Cairo and cold along the Mediterranean coast.

Between March and April the outstanding phenomena is the Khamassin, a dry hot and often dusty wind that can blow in from the parched Western Desert at up to 150 km per hour.

Climate parameters for Qena and Sohag located some 65 km up and downstream of the site respectively, are available from the Meteorological Authority, Climate Department. Monthly rates for selected parameters for Qena and Sohag are shown in **Table 2.2**.

**Table 2.2: Selected Climatic Parameters at Site**

| Parameter | Qena | Sohag | Unit |
|---|---|---|---|
| **Air Temperature** | | | |
| Max. Air Temperature | 41.2 (June) | 39.3 (June) | °C |
| Min. Air Temperature | 6.9 (Jan) | 7.2 (Jan) | °C |
| **Rain** | | | |
| Average of yearly rainfall | 3.6 | 2.2 | mm |
| Max. of monthly average rainfall | 0.9 (Nov) | 0.6 (Feb) | mm/month |
| Max. of one single rainfall | 55 (Nov) | 14.6 (Apr) | mm/day |
| **Barometric Pressure** | | | |
| Max. barometric pressure at sea level | 1031.9 (Jan) | 1033.7 (Jan) | HPa |
| Min. barometric pressure at sea level | 998.8 (Aug) | 1001.3 (May/Aug) | HPa |
| **Relative Humidity** | | | |
| Max. at 3.00 International Time | 61(Dec) | 67 (Jan/Dec) | % |
| Min. at 3.00 International Time | 32 (May) | 40 (May) | % |
| Max. at 12.00 International Time | 31 (Dec) | 43 (Jan/Dec) | % |
| Min. at 12.00 International Time | 16 (May/Jun) | 3 (Apr) | % |
| Overall Minimum | 1 | 3 | % |

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Monthly wind rose data for Qena and Sohag is available from the Meteorological Authority, Climate Department. Selected data are summarised in **Table 2.3**.

### Table 2.3: Wind Data of Project Area

| Prevailing Wind | Qena | Sohag | Unit |
|---|---|---|---|
| Direction* | 270 (all months) | 330 (all months) | |
| Max. Speed | > 21 (May) | 11-16 (Jan to May) | m/s |

* 0 = 360 = North, 90 = East, 180 = South, 270 = West

## 2.5 Natural River Water Conditions at Site

### 2.5.1 Water Temperature

Max. pool temperature at surface in summer:   30° C
Min. pool temperature at surface in winter:   15° C

### 2.5.2 Suspended Load

The silt content of the water flowing through the turbine is expected to be in average between 1 and 50 ppm or $g/m^3$ with peaks up to 100 ppm or $g/m^3$ during high flood periods up to 2,400 $m^3/s$.

### 2.5.3 Silt Analysis

| | |
|---|---|
| Fine sand | $D_{16} = (0.137 \text{ mm})$ |
| Medium sand | $D_{50} = (0.380 \text{ mm})$ |
| Coarse sand | $D_{84} = (1.123 \text{ mm})$ |
| Average silt density | $\rho = 2,859 \text{ kg/m}^3$ |

### 2.5.4 Water Quality Data (Chemical Analysis)

### Table 2.4: Chemical Analysis of River Nile Water at Naga Hammadi Barrage

| Parameter | Formula | Unit | Sample | | |
|---|---|---|---|---|---|
| | | | 1993 | Feb. 1999 | Feb. 1999 |
| Total filterable residue dried | TDS | ppm | 310 | 420 | 390 |
| Alkalinity as Sodium Carbonate | $Na_2CO_3$ | ppm | 170 | 159 | 170 |
| Salinity as Sodium Chloride | NaCl | ppm | 59 | 59 | 59 |
| Salinity as Chloride Ion | $Cl^-$ | ppm | n.a. | 36 | 36 |
| Sulphate as Sulpher Trioxide | $SO_3$ | ppm | 29 | 122 | 78 |
| pH-Value | | | 7.0 | 7.0 | 7.0 |

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# CHAPTER 3

# PLANNING CRITERIA FOR THE NEW BARRAGE

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# 3. PLANNING CRITERIA FOR THE NEW BARRAGE

## 3.1 Introduction

The development of planning criteria has been evolving since the feasibility study. Overall, the planning criteria applied for the New Barrage are an amalgam of results from the engineering studies and field investigations during both the Interim and Feasibility phases, technical recommendations by the PoE made during the feasibility phase, the results obtained from hydraulic model investigations performed after the feasibility study, and the operational requirements finally introduced by the MWRI.

The planning criteria were completed by the results of inquiries with the PIU and by the outcome of inspections of the New Esna Barrage structures. The site conditions for the New Naga Hammadi Barrage differ substantially from those at Esna, thereby requiring a different approach for construction.

## 3.2 Headpond and Design Floods

### 3.2.1 Headpond Level

The headpond level of the New Barrage shall be at 65.90 m asl, which is equivalent to the design level of the existing Barrage. This level is about 0.50 to 0.80 m above the headpond levels experienced at the Barrage during the recent years.

The MWRI further stated that during operation the headpond level of the New Barrage will be maintained at 65.90 m asl for at least 90% of the time, but during some 30 days per year the headpond level may be lowered within a range of up to 0.5 m. This temporary, short-term reduction is proposed to meet short-term increases in the irrigation demand downstream of Naga Hammadi. The significant distance from the HAD to the irrigation offtakes serving the areas of increased demand, and hence time lag in water reaching these areas, precludes they being met quickly by an increased release from the HAD.

### 3.2.2 Design Flood

Design floods to be safely evacuated by the sluiceway of the New Barrage will be:

- the 1:10,000 year flood routed through Lake Nasser and released from the HAD at a flow rate of 5,700 m$^3$/s, and

- the 7,000 m$^3$/s emergency flood releases from the HAD.

For the 1:10,000 year flood, a flood surcharge above the normal operating level (NOL) of the headpond shall not occur.

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### 3.2.3   Limitation of Reservoir Surcharge During Emergency Flood

It is a requirement that the flood level during the emergency discharge should not rise above historic flood levels experienced in the years before commissioning of the HAD, especially those levels occurring in 1959 and 1960, years of high floods. The MWRI has limited the maximum headpond surcharge at the New Barrage to 67.40 m asl, 1.5 m above NOL. With the recently developed shape of the sill and the adjacent apron and concrete slab, the headpond level does not rise during the 5,700 $m^3$/s release. The model tests further reveal that with the 7,000 $m^3$/s emergency release and all gates withdrawn, the headpond level rises only to 67.05 m asl. With only 6 gates fully open, the headpond level rises to 67.24 m asl. Hence, the additional capacity of one of the navigation lock chambers will not be needed.

### 3.2.4   Freeboard Allowance and Crest Level of Main Barrage Structures

The freeboard of the main barrage structures shall allow for:

- Water surface set-up by wind,

- effect of wave run-up on sloping embankments or wave breaking on vertical walls, and

- surges as a result of load rejection from the powerplant

with an additional safety margin.

The safety margin for normal operation, including the surges from load rejection, shall be more than 1.5 m, whereas for the safety margin during the emergency release, 1 m is considered acceptable.

From detailed calculations at feasibility study level and the safety margins, the crest elevation of the main barrage components is 69.00 m asl.

## 3.3   Flood Evacuation and Sluiceway

### 3.3.1   Width and Number of Sluiceway Gates

The gate width of the New Barrage sluiceway is equivalent to that of the navigation locks (17.0 m) to allow interchanging of the stoplogs, if necessary.

With the 7,000 $m^3$/s emergency release being the maximum discharge to be evacuated by the sluiceway and with the associated limitation of the headpond surcharge, the number of gate openings was determined to be seven.

### 3.3.2   Additional Safety for Evacuation of Emergency Release

The sluiceway with all gates open shall have the capacity to evacuate the emergency release of 7,000 $m^3$/s without participation of the navigation lock. However, the MWRI requires as an additional safety for releasing the emergency discharge that one of the navigation lock chambers and its upstream and downstream approaches are designed for participation in flood evacuation. Preparation of one of the navigation locks for flood evacuation requires a type of gate at the up-stream portal which is able to operate under flood conditions, e.g. a segmental gate.

The second navigation lock chamber does not need to be prepared for participation in evacuation of the emergency flood, as the first chamber already satisfies the increased safety requirements.

### 3.3.3 Sluiceway Gates

All sluiceway gates shall be equipped with a flap. The release capacity of all flaps together shall be sufficient to release the river discharge exceeding the capacity of the turbines (1,670 m³/s) during the months of peak irrigation demand. This ability to operate the sluiceway avoids the need to lift the main gates during the high flow season or when small differential discharges between powerplant and river flow occur. From hydraulic model tests, the capacity of the flaps at a headpond level of 65.90 m asl resulted with 800 m³/s.

If the flow to be discharged over the sluiceway exceeds 800 m³/s, the radial gates shall be used. The minimum number of gates to be opened at a time is three, minimum gate opening is 1.20 m. The related discharge capacity is 710 m³/s, as determined in hydraulic model tests. If the powerhouse is out of operation, the three gates can be opened up to 3.00 m before additional sluiceway gates have to be opened due to dangering the riprap stability. The related discharge is 1,430 m³/s. For discharges above 1,430 m³/s (without powerhouse) at least 6 gates shall be operated simultaneously.

### 3.4    Minimum River Discharges

The re-distribution of the releases from the HAD over the year as already observed during the years 1994, 1995 and 1996 was regarded to be indicative of the future release pattern from the HAD. The minimum release during the winter closure period was decreased from 710 to 350 m³/s. This has to be considered in the setting of the bottom of the navigation lock and the approaches, and in the setting of the generating units.

### 3.5    River Diversion and Navigation

### 3.5.1    Diversion Flood Discharge

The diversion flood discharge shall be 2,900 m³/s. This corresponds to a release from the HAD of a flood inflow to Lake Nasser with a recurrence interval of 1:100 years and constitutes a high safety for the construction stage of the New Barrage.

The same river discharge of 2,900 m³/s is specified by the General Authority of Nile Transport (GANT) as being the maximum navigable river discharge.

### 3.5.2    Continued River Navigation

During construction river navigation through the site of the New Barrage shall not be interrupted, but some restrictions to navigation can be imposed during the "closure period" of the irrigation systems supplied by the River Nile. One of the navigation lock chambers must be passable at unimpounded headwater levels during the construction of the closure dam, but limited to a flow minimum rate of 710 m³/s. For this purpose, the sill of the upstream portal of one of the navigation lock chambers shall be sufficiently low to guarantee the required water depth (keel clearance of 0.5 m plus draught 1.8 m). This requires that the upstream forebay be provided to sufficient depth, e.g. to elevation 56.60 m asl.

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Neither the chamber of the second navigation lock nor the approaches have to be dimensioned for continued river navigation during the period of unimpounded headwater.

Both during construction and after implementation of the project the maximum discharge under which navigation must be possible is 2,900 m$^3$/s. The minimum discharge for navigation is 350 m$^3$/s, but during construction reduced keel clearance or reduced permissible draught of vessels sluicing can be accepted for discharges lower than 710 m$^3$/s.

### 3.5.3  Limitation of the Width of the Diversion Canal

For the period of river diversion during construction of the New Barrage, the GANT has requested a minimum navigation path width of 100 m for safe navigation in the diversion canal, considering that two vessels have to pass each other. The width of the diversion canal varies with distance from the river section where it branches off the main river. The shaping, width and alignment of the diversion canal is the outcome of the hydraulic model tests. Its minimum bottom width is 125 m. The entire diversion canal requires to be riprap protected, however, the need for protection is limited for the period of construction.

### 3.5.4  Temporary Works

The cofferdams of the construction pit together with the seepage cut-off wall are considered as temporary works, and their execution and maintenance during the period of construction including their removal shall be considered as being at the risk of the contractor.

The design criteria given on such works shall be considered as preliminary and may be subject to adjustments by the selected contractor for the civil works.

### 3.5.5  Seepage Cut-off Wall for Construction Pit

From the results of the borehole investigations, it was concluded that the construction pit must be enclosed by a deep seepage cut-off ring wall which intersects permeable layers of gravel and sand at depth. Connection to the confirmed lower seal layer of clay should be achieved by keying into the clay layer with a minimum depth of 3 m. The seepage cut-off ring wall shall be entirely completed before commencement of dewatering of the construction pit below the prevailing river water level.

### 3.5.6  Height of Cofferdams for Construction Pit

The crest level of the cofferdams for the construction pit shall be well above the artesian head encountered during geotechnical investigations in the feasibility study. This avoids during construction that flow develops through the trench of the cut-off wall from layers under artesian pressure.

The crest levels of the cofferdams for the construction pit shall be dimensioned with a safety margin of at least 1.0 m above the 2,900 m$^3$/s river water level under the changed river hydraulic conditions with the diversion canal.

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### 3.5.7 Single-Stage Construction Pit

As a result of previous studies comparing single and staged construction pits, it is concluded that under the prevailing site conditions and in view of the considerable depth of the pit for the powerhouse foundation, only the single stage construction pit is viable.

### 3.5.8 Safety of Construction Pit against Uplift and Boiling

The factor of safety against uplift of the dewatered construction pit encased by the cut-off wall and the clay layer shall be at least 1.10 according to DIN 1054 and EAB. The factor of safety required against boiling for sand (< 0.2 mm) and silt is 2.0.

## 3.6 Navigation Locks

### 3.6.1 Number of Lock Chambers

The PIU took the decision that a second navigation lock chamber shall be built at the side of the originally proposed single chamber, and with the same length. The operation of each navigation lock chamber shall be independently possible. There is no need for saving of water.

### 3.6.2 Dimensions of the Navigation Locks

As for the new navigation locks at the New Esna and Naga Hammadi Barrages, the GANT requires that the dimensions of the two navigation locks to be constructed at the New Barrage are based on the following requirements:

- Net length        160 m
- Net width         17 m

The net length of a navigation lock chamber is measured between upstream and downstream limitation marks of the usable chamber length.

With the present riverbed conditions, the level of the downstream sill shall allow for a water depth of 3.0 m below water level at minimum flow of 350 m$^3$/s. In this depth, the predicted decrease in downstream water levels due to future riverbed degradation is already considered. The adopted floor elevation of the downstream approach and lock chamber is 54.90 m asl.

The filling/emptying system of the navigation lock chambers shall consist in short ducts with a dissipation chamber of minimised dimensions in the area which is surpassed by the mitre gate wings at the downstream portals. The ducts shall each have one upstream and one downstream regulating gate, together with stoplog slots on both sides of the regulating gate for maintenance.

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### 3.6.3 Directional River Traffic

Each of the navigation lock chambers shall be operated under normal conditions for unidirectional traffic only, e.g. the left chamber for traffic in upstream and the right chamber for traffic in downstream direction. Berthing areas are permitting that two ships in each direction can remain in waiting position. The waiting area for the downstream traffic direction is arranged on the bank side whereas the waiting area for upstream traffic is on the guidewall side.

For the case that the left navigation lock chambers remains closed for maintenance, a second waiting area is provided downstream and near the right bank. This is necessary for manoeuvring under the restricted conditions in the downstream approach bay.

### 3.6.4 Safety Devices of the Navigation Lock and Permissible Hawser Stresses

The downstream portals shall be protected against impacts from ships entering the lock chamber, by cable restraining devices which can be lifted. The upstream portals require no arrester systems if the crest level of the upstream sill is sufficiently near to the 2,900 m³/s tailwater level.

The safety distances between the cable restraining devices and the marks indicating the usable length of the lock chamber shall not be less than 3 m.

The hawser stresses on a ship shall not exceed 50 kN.

## 3.7 Public Traffic and Access for Operation and Maintenance

### 3.7.1 Separation of Public Traffic from Operational Areas on the Barrage

Separation of the main road for public traffic and the area served by the main gantry cranes for operation and maintenance is required from the viewpoints of safety and cross-river traffic. The main gantry cranes serve the entire length of the powerhouse and sluiceway, except the downstream portals of the navigation lock. The latter is not necessary as both lock chambers shall be served by separate gantry cranes, which is a requirement of the MWRI.

The downstream stoplogs of the turbine draft tubes are operated by a gantry crane on a runway separate from the public road.

There shall be no permanent access from the public road to the operational areas.

### 3.7.2 Service Bridge on the New Barrage

The sluiceway section of the New Barrage shall have a service bridge, the size of which depends on operational requirements, e.g. to pass by vehicles from the powerhouse platform over the sluiceway to the platform of the inlet and outlet gates of the navigation lock filling/emptying ducts. The service bridge shall be designed for access by small vehicles (5 t). Access from the right bank to the intermediate platform between sluiceway and the first lock chamber is by a walkway over the navigation lock gates.

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### 3.7.3 Public Road Bridge

The PIU decided that a public road bridge shall be constructed on the New Barrage to be commissioned together with the barrage structures, and shall be designed as an elevated road bridge.

The elevated road bridge shall consist in an asymmetrical structure with ramp on the sluiceway to a high level fixed bridge crossing the two navigation locks. On the west side of the navigation lock, a concrete and embankment ramp will descend to normal road level. The clearance for any fixed structure over the navigation lock is required to be 13 m above the maximum navigable tailwater level.

## 3.8 Hydropower Plant

### 3.8.1 Power Plant Operation

With the aim of avoiding any impact on the control of the irrigation system, the MWRI and the MEE agreed that the powerplant is operated only in the run-of-river mode.

### 3.8.2 Installed Capacity

In the feasibility study, the installed capacity was determined by economic criteria, in which the incremental energy cost was limited by the cost of equivalent thermal generation. The optimisation resulted in an average annual energy of 463 GWh. Adjustment of the calculations resulted in a limitation of the maximum turbine release to 417.5 m$^3$/s and a powerhouse design discharge of 1,280 m$^3$/s. This corresponds to 53 % exceedance in the annual duration curve.

The optimisation leading to the average annual energy of 463 GWh was based on a powerhouse with four generating units with three blade bulb turbines. It was further shown in the feasibility study that the extraction of discharge from Lake Nasser by the Mubarak (former Toshka) pumping station has no significant effect on the overall energy generation to be expected at Naga Hammadi, nor on the design data of the generating units.

### 3.8.3 Type and Number of Generating Units

Double regulated bulb turbines with directly driven generators are selected for the head and flow conditions in the River Nile at Naga Hammadi. With the aim of minimising cost and maximising energy generation, both three and four blade bulb turbines were under investigation. While the four blade units reach generally higher efficiencies over the entire range of operation, their discharge capacity above design discharge is significantly smaller than that of a three blade unit of the same diameter. With the same diameter and at lower cost, the three blade units can operate over a larger range of the discharges during the high flow period than would be possible with four blades. As the energy generation with the three blade units is higher than with four blade units of identical size the three blade bulb turbines were selected.

Incremental cost of generation over a range of discharge capacities in the optimisation revealed production cost increments for a six unit powerhouse being clearly above critical limits whereas the production cost for a four unit powerhouse was acceptable.

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### 3.8.4 Bulb Turbine Rating and Setting

The four turbines are designed for the following operating conditions and will not be re-optimised:

- Total powerhouse design discharge, 4 x 320 m³/s: $Q_t$ = 1,280 m³/s

- Maximum powerhouse discharge, 4 x 417.5 m³/s: max $Q_t$ = 1,670 m³/s

- Corresponding net head with no flow over the sluiceway: $H_n$ = 4.20 m

- Maximum net head at 350 m³/s powerhouse discharge with no flow over the sluiceway: max $H_n$ = 8.75 m

- Minimum net head for operation in the network during flood conditions, corresponding to a river discharge of approximately 3,520 m³/s: min $H_n$ = 2.40 m

A potential increase of the net head up to 0.8 m due to future degradation of the downstream riverbed is taken into account when setting the turbine centre line. The maximum turbine power under potential future increase of the net head in 50 years is taken into consideration when rating the generator. The maximum increase in turbine output which could occur in the following 50 years will still be acceptable for generators with class F insulation and a decrease in power factor from 0.85 to 0.90 within the economic lifetime of the project. EEA has no objection in correcting the power factor in the long run to accommodate the possible increase of capacity of the New Naga Hammadi Barrage Hydropower Plant.

### 3.8.5 Surges in the Headpond

Hydropower operation could induce surges in the headpond and the tailwater. Critical surges would result from the case of transmission failure, after which the sluiceway would have to be fully operational within six minutes. During the critical period until the sluiceway assumes the full powerplant discharge, releases from the powerplant would constitute 50% of the total for approximately two minutes. Maximum surge height shall not exceed 0.30 m directly upstream of the structure.

### 3.8.6 Type of Switchgear at the New Barrage

The conventional high voltage outdoor switchgear type was selected by HPPEA/EEA due to maintenance procedures common to their other plants in the system. On request of HPPEA, the 11 kV/220 kV transformers shall be located outdoors at the switchyard. They are connected via some 400 m long 11 kV cables with the generators in the powerhouse.

### 3.8.7 Transmission Voltage between the New Barrage and Naga Hammadi Substation

In view of the results of EEA's network planning, EEA decided that a transmission voltage of 220 kV is applied to the double line.

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## 3.9    Control of Barrage and Powerplant Functions

Control and monitoring of the New Barrage project will be divided between two organisations, the EEA being responsible for the operation of the hydropower plant and the MWRI being responsible for the total control of releases from the HAD to the downstream reaches of the River Nile, the control of the headpond level which affects the position of the head regulators at the irrigation canals, and finally the control of the navigation locks.

For the hydropower plant it was requested by EEA that the control room be located in the powerplant. Control data will also be displayed in EEA's administration building.

The operation of the sluiceway gates will be linked to the operation of the hydropower plant for the following two cases:

- during the period when river discharges exceed the powerplant discharges,

- in case of load rejection due to failure of plant or transmission, the sluiceway gates must be opened automatically within six minutes.

Direct operation of the sluiceway gates and flaps will be the responsibility of the MWRI. The MWRI control room will also contain a display of the headpond levels and powerplant discharges (linked to EEA's data acquisition system).

## 3.10    Riverbed and Bank Protection against Concentrated Barrage Releases

### 3.10.1  General

For the areas which need additional protection against higher flow velocities by riprap, such as those in the vicinity of the structures (intake and tailrace), the thickness of protection and range of riprap stone size are given for different flow velocities by the hydraulic model tests.

### 3.10.2  Areas affected by the Sluiceway Operation

From the results of the hydraulic model tests with riprap of different classifications, riprap and filter classes are defined for site conditions with corrective factors on the specific density of the riprap material (low density limestone).

Heavy riprap protection shall be as far as possible limited to the area of the construction pit, where the temporary sealing element (e.g. concrete diaphragm wall) forms a limitation towards a lighter outside protection. Within the concrete diaphragm ring, the protection shall withstand critical releases involving supercritical flow under the partially opened gates, up to the 1:10,000 years release.

The overall extent of the riverbed protection outside of the concrete ring shall be limited to the attack of the 1:100 years flood, however, the thickness of protection and the typical diameters of riprap shall be for the 1:10,000 years flood discharge.

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### 3.10.3  Limitation of Sluiceway Operation regarding Riverbed Protection

Hydraulic investigations have shown that a single gate shall not be opened at any tailwater condition. This leads to obstruction of the riprap protection downstream of the apron, even of the heaviest type of protection. The operation of the gates shall be normally limited to simultaneous lifting of all gates with the same lift. However, it should also be possible to lift only a group of gates for a certain lift height depending on the tailwater conditions, without the risk of downstream scouring. The limited range in which this operation of a group of three gates only is possible, was defined by additional hydraulic model tests.

### 3.10.4  Protection of Powerhouse Approach and Exit Bays

Riprap stability related to the powerhouse was investigated for the adapted tender design in additional hydraulic model tests.

Approach flow was investigated for operation of all four units at maximum turbine discharge (total 1,670 m$^3$/s). Special attention was to be given to the protection of the piers, both the abutment and the intermediate pier between powerhouse and sluiceway.

Riprap stability of the downstream of the powerhouse was to be proven for the critical case of lower tailwater conditions as estimated after 50 years of riverbed degradation and operation of all four units at maximum discharge.

Sufficient safety of the riprap stability is important once the riprap is more or less permanently exposed to the exit flow velocities with a maximum of 2.7 m/s.

Hydraulic model tests have shown that riprap protection can directly start at the exit section of the draft tube, without a concrete transition apron. The type of riprap which has shown to be stable under full turbine and minimum tailwater conditions (350 m$^3$/s) has a mean diameter of 0.74 m (riprap type R1).

### 3.10.5  Protection of Riverbanks

Protection of the riverbank is required especially in the area immediately downstream of the barrage. The size of adequate riprap material resulted from hydraulic model tests. In addition wave actions and propeller jets of the ships were considered for a stable river bank design. The standardised riprap protection with adequately designed filter layers was compared with alternative solutions, such as using mats and colloidal mortar to bind the upper layer of stones in order to reduce quantities and thickness of the layers. However, it was found that for Egypt and the particular project conditions, the proposed design is most appropriate.

### 3.10.6  Protection of Navigation Lock Approach Bays

The protections of the navigation lock approach bays are dimensioned for the low flow velocities prevailing on both approaches of the locks. The impact of propeller jet will be small in the upstream approach with 0.5 m heel clearance. The same effect in the downstream approach bays will be significant at minimum tailwater and leads to the criteria of riprap dimensioning.

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Normal dimensions of riprap are locally and drastically increased by the required ability of the left lock chamber to participate in the evacuation of the emergency release. As the passage of the flood discharge will be under subcritical conditions and with an approximate average flow velocity of some 4.5 m/s in the lock section, the riprap of the approaches some 350 m upstream and some 400 m in downstream within the surroundings of the diaphragm wall shall be of the same type as immediately downstream of the sluiceway. Local strengthening of the protection at both inlet and outlet sections resulted from additional hydraulic model tests.

## 3.11 Head Regulators

The head regulators for the eastern and the western canal shall be rehabilitated. They consist of the same materials as the old Barrage, e.g. rubble masonry piers and arches with dressed sandstone cladding on a foundation of unreinforced concrete, corresponding to compressive strength values in the range of 10 to 20 MPa. As from investigations it resulted that the structures are fully intact except the platform filling of the eastern regulator which suffered from heavy traffic, rehabilitation is justified.

The pre-HAD function of releasing flood discharges through the head regulators is no more needed. Hence, the required numbers of gates of the old double leaf gates could be reduced to discharge the present maximum irrigation requirement plus an addition of 4% to take into account the future planned expansion of irrigation areas supplied from the canals.

This results in a permanent closure of the upper gate leaf openings in both head regulators. In addition, in the western regulating structures two of the lower gate leaf openings could be permanently closed. However, the MWRI requested to maintain all six openings and to rehabilitate all six gates. The remaining flow sections in each regulator must suffice to pass the present maximum discharge when one of the remaining gates is out of operation.

The rehabilitation of the pier and arch structures, e.g. by void grouting, can be carried out at any time. Rehabilitation of the downstream aprons can only be performed during the time of winter closure of the irrigation system, e.g. in December and January each year.

The present live load to be supported by each structure's platform is a 30 t vehicle. This load limitation shall be maintained.

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# CHAPTER 4

# ARRANGEMENT OF THE NEW BARRAGE STRUCTURES

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# 4. ARRANGEMENT OF THE NEW BARRAGE STRUCTURES

## 4.1 Location and Layout

Based on the results of geotechnical borehole investigations, it can reasonably be assumed that the future construction pit is completely underlain by a continuous clay layer at depth, with a thickness varying between 6 and 12 m. This was the primary reason governing site selection. Another borehole campaign was carried out to confirm these conditions and to complete the knowledge about soil conditions along the proposed diaphragm wall around the construction pit.

The barrage components and their sequence of integration into the New Barrage layout are shown in plan in **Album Nos. 6** and **7**, and in section perpendicular to the river axis in **Album No. 71**. The combination of the main concrete structures shown on the drawings is similar to that developed in the feasibility study. With the aim of minimising interference to navigation resulting from the approach flow to the hydropower station, the sluiceway is located between the hydropower plant and the navigation lock. The sluiceway is expected to be operated for only relatively short periods during the three high-flow season months (June to August) with discharges up to some 800 m³/s, or with the full river discharge at times when the powerplant does not operate.

The sluiceway and the hydropower plant are located within the original course of the river. Aiming to achieve a regular distribution of flow velocities of the New Barrage as near downstream as possible, the powerhouse is located on the left side of the river. It is suggested that this arrangement minimises impacts on river morphology since during the major periods of the year, only the powerhouse operates. Even in the three summer months, operating together with the sluiceway, the powerhouse exit flow is more concentrated than the release from the sluiceway. As a result, the navigation lock is located on the right bank, providing for a safe approach to and release from the navigation lock (**Album No. 6**).

The three structures are separated by large piers, which serve to separate the different floor levels of the approaches to the structures. For the hydropower intake, sluiceway, and navigation locks the approach floor levels are 41.00 m asl (lowest), 49.10 m asl, and 56.60 m asl respectively. All piers are compact reinforced concrete structures, which are partly filled with gravel to increase weight with the internal water level adapting to the river water level.

The navigation locks and the sluiceway are separated by the sluiceway abutment pier with an adjacent connecting structure which is used to accommodate the filling and emptying ducts of the navigation lock (intake is from the headpond and outlet to the tailpond) and the operating and service gates. The two navigation locks are located on the right side of the sluiceway, with the upstream and downstream approaches cut into the banks of El Dom Island. The location of the downstream portal is as near as possible to the road bridge, with the aim of reducing construction quantities and costs.

The hydropower plant, which requires the lowest foundation level, is separated from the sluiceway by the intermediate pier (**Album Nos. 7** and **35**) which has the lowest foundation level of 38.10 m asl. The shape of the upstream end of this pier is important in maintaining adequate approach flow conditions to the powerhouse when operated separately or in combination with the sluiceway. The shape and the protection of the riverbed has been tested and adjusted during the hydraulic model investigations.

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To the left, the powerplant ends with the abutment pier. This absorbs the earth pressure from earth fill used to form the unloading platform adjacent to the small artificial island remaining from the original west bank of the river after construction of the diversion canal.

The riverbed at this location is narrow and river diversion during construction will require a separate temporary diversion canal on the west bank of the River Nile.

A number of aspects regarding access conditions influenced the layout and the design:

- Operationally and for maintenance and access with loads to the sluiceway and powerplant, it is necessary that 2 gantry cranes are able to travel along the entire length of both structures from the unloading platform between the diversion canal and powerplant. The crane beams are placed on a level a short distance to the water surface, which does not permit the main gantry cranes to serve the downstream portals of the navigation locks. The crane rails extend onto the sluiceway abutment pier to load or unload equipment from the navigation locks which is moved to the pier platform on carriages, e.g. stoplogs, etc.. There is one gantry crane for loads of up to 15 t, travelling along each of the lock chambers. On the central platform between the two navigation lock chambers there is another provision to move loads (stoplogs) between both gantry cranes on a carriage.

- The design of the New Barrage shall satisfy two conditions of access, one for service and operational purposes, and the other for public traffic with appropriate road connections. The service bridge for small vehicles shall span the sluiceway only.

- Public traffic must remain uninterrupted during operation and maintenance procedures involving the main gantry cranes. Hence, separation of the public traffic road from the runway of the cranes is required. This condition is met by locating the crane runway upstream and the public road downstream above the draft tubes of the powerhouse. The spacing of the crane beams is determined by the width of the main powerhouse hall (see **Album No. 29**). The space between the upstream and downstream stoplogs of the sluiceway is coordinated with the beams of the crane runway.

- For hydraulic reasons, the downstream stoplogs shall be located at the end of the draft tubes, which require a downstream service crane beam bridge separate from the public road bridge.

From the left bank, the public road passes the closure embankment and continues in a straight line over the main concrete structures. From the public road on the closure embankment, there is access to the switchyard and to the administration buildings and workshops of both Employers, the MWRI and the MEE/HPPEA. From the workshop areas on the backfill of the upstream section of the diversion canal, there is a road connection to the paved unloading area on which the crane rails end. On the powerhouse, the total concrete platform except the public road is available for operational access. Between the intermediate and the sluiceway abutment piers, there is the service bridge with a 3 m wide platform to be used by small vehicles up to 12 tons. The bridge remains horizontal over the sluiceway width and contains the trays for cable connections between both river banks.

The service bridge provides access to the platform of the inlet/outlet structure for the navigation lock filling/emptying systems. Access between the navigation locks and to the right bank is only for pedestrians via the mitre gates. There is an access from the public road bridge to the central platform via the staircase shaft (elevator) with guard tower.

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For the high level public road crossing of the navigation lock, a clearance of 13 m above maximum navigable water level at 2,900 m³/s is required. Road crossing of the navigation lock is only viable downstream of the downstream portal. The road must be elevated by ramps on both sides of the navigation lock, one ramp being on the sluiceway and supported by elevated piers, and the other ramp formed by an embankment on the right abutment.

## 4.2    Summary of Design Water Levels in the Future Headpond

River discharge conditions and related headpond level are summarised in **Table 4.1**.

### Table 4.1: New Barrage Functions

| River Discharge Conditions | Discharge Rate m³/s | Barrage Function | Headpond Level m asl |
|---|---|---|---|
| Range of normal discharge | 350 - 1,670 | - Irrigation supply<br>- Hydropower generation<br>- Sluicing for navigation<br>- All sluiceway gates closed | 65.90 |
| Range of high irrigation demand | 1,670 - 2,400 | - Irrigation supply<br>- Hydropower generation<br>- Sluicing for navigation<br>- Sluiceway flap gates operating | 65.90 |
| Range of high river discharge (upper limit exceeds $Q_{100}$) | 2,400 - 2,900 | - Irrigation supply<br>- Hydropower generation<br>- Sluicing for navigation (upper limit)<br>- Sluiceway gates operating | 65.90 |
| 1:10,000 year flood discharge | 5,700 | - Curtailed irrigation supply<br>- Hydropower plant closed above 3,520 m³/s<br>- Navigation lock closed<br>- All sluiceway gates fully open at 5,700 m³/s<br>- No headpond surcharge | 65.90 |
| Emergency discharge (release from HAD) | 7,000 | - Hydropower plant closed<br>- Navigation lock closed<br>- All sluiceway gates open | 67.05 |

## 4.3    Temporary Works

### 4.3.1  Construction Pit

Prior to construction of the main concrete structures, the construction pit must be excavated. The size and protection of the construction pit remains within the responsibility and liability of the Lot 1 Contractor, and the following descriptions include the approach which the Consultant would adopt.

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The construction pit will extend across the entire width of the riverbed plus limited portions of both river banks. **Album No. 18** shows the arrangement of the construction pit with the diversion canal while a more detailed plan of the pit is presented in **Album No. 19**. The extension of the construction pit onto El Dom Island is determined by the length of the navigation locks, including the upstream concrete guide wall. On the west bank, there remains a part of the former bank between diversion canal and pit.

The lateral extent of the construction pit is determined by the allowable excavation slopes during construction, which are in turn based on the properties of the surrounding sand. With embankment slopes of 1:2.5 and an excavation depth of some 13 m below existing ground levels, the total width of the construction pit is some 530 m while the length at the side of the navigation lock extends to around 580 m. The total quantities to be excavated depend largely on the depth and extent of the existing river channel but are estimated to be approximately 1.8 million m$^3$.

The upstream and downstream boundaries of the construction pit are formed by cofferdams up to 14 m in height. When the diversion canal is completed, construction of the cofferdam will commence with end-tipping of rockfill from the riverbanks or dumping in the river from barges, followed by filter material and sandfill. **Album No. 21** indicates the order of placement with the second stage sandfill being on the upstream side of the first stage rockfill. This will ensure that sandy material penetrates the rockfill and supports the second stage sand embankment.

A diaphragm wall is required to ensure adequate safety against hydraulic failure when dewatering. Layers of sand and gravel of high permeability, which can be expected to transmit a significant amount of seepage under artesian conditions, would otherwise make this task difficult. The platform for the preparation of the diaphragm wall must, however, remain well above the hydraulic grade line of artesian pressure which extends from the headpond level downstream.

The method of establishing the diaphragm wall will be by slurry trenching with cutter panels. Although some risks remain, this approach is recommended based on lower artesian pressures observed in this area, and the comparatively higher progress rate achievable by trenching during construction.

The diaphragm wall shall be keyed some 3 m into the clay layer which was encountered during drilling at an average elevation of 15 m asl. Based on the grid of boreholes drilled so far, it is assumed that this layer underlies the entire area of the construction pit.

The diaphragm wall is proposed to have a constant thickness of 1.0 m, penetrating both cofferdam and riverbed material. It will enclose the entire construction pit. The diaphragm wall section along the right bank will be taken as part of the permanent works, described in **Section 4.4.5**. In view of the proposed seepage cut-off to the construction pit, the estimated inflow will depend largely on the quality of the diaphragm wall but is expected to be minimal.

With the tight encasing of the construction pit, uplift under the clay layer can be critical when excavating and dewatering the construction pit to the lowest foundation level. Calculations have indicated that a sufficient factor of safety can be maintained at the minimum foundation level required for the powerhouse and pier foundations, which is 38.10 m asl, some 25 m below the water level during passage of the diversion flood.

Access to different levels of the construction pit during construction is provided by a system of ramps on the inner slopes of the excavations, accessible from the side of El Dom Island.

## 4.3.2   Diversion Canal

Minimising the excavation quantities for the diversion canal requires that the canal bends around the shorter side of the pit where the powerplant is located constituting as well the preferred alignment for hydraulic reasons.

The diversion canal extends to the west side of the construction pit and has an approximate length of some 1,100 m and a base at 52.0 m asl. During the minimum navigable river discharge of 350 $m^3$/s, the flow depth will be 5.9 m.

In view of the temporary nature of the diversion canal, its width was minimised to reduce cost. The minimum width at the bottom is between 125 and 135 m. Average flow velocities admissible during the four years of construction may increase by some 20% above maximum average flow velocities under natural conditions in the River Nile, which in this river section are 1.3 m/s.

Criteria considered in designing the diversion canal were:

- Velocities transverse to the navigation route must not exceed 0.3 m/s.

- Navigation shall not be impaired for all river discharges up to 2,400 $m^3$/s, and still be possible for discharges up to 2,900 $m^3$/s through the diversion canal. Discharges exceeding 2,000 $m^3$/s occur over only 3 months of the year from June to August.

- Erosion criteria limit maximum local velocities in the riprap-lined canal for the smallest riprap types and without application of geotextiles.

Maximum velocities occurring during operation of the diversion canal depend on the canal alignment and the shape of the cofferdam around the construction pit. Upstream, the velocity distribution depends on the velocity profile of the river flow at the canal inlet. Downstream, the velocity distribution is affected by the transition to the natural river cross-section. The shaping of the diversion canal was performed by hydraulic model tests in the Barrage Model, see **Chapter 5**.

## 4.4     Permanent Main Structures

### 4.4.1   Sluiceway

The sluiceway structure, shown on **Album Nos. 37** to **42**, was dimensioned for a capacity of 5,700 $m^3$/s with full opening of all gates and maintaining the headpond level at 65.90 m asl. This discharge corresponds to the release from the HAD of an inflow flood to Lake Nasser with a 1:10,000 year recurrence interval. Despite of this conservative design basis, there is a requirement imposed by the MWRI that the barrages in Upper Egypt shall be able to pass an emergency release of 7,000 $m^3$/s from the HAD.

It was required by the MWRI that the New Barrage shall be able to evacuate the 7,000 $m^3$/s by the sluiceway only, with full opening of all gates, whilst the headpond level rise shall be limited to 67.40 m asl corresponding to pre-HAD flood marks.

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The sluiceway of the New Barrage consists of 7 bays, each equipped with a 17 m wide radial gate. The hydraulic design of the sluiceway of the New Barrage is similar to that of the existing sluiceway at the New Esna Barrage. However, the sill and apron levels and the length of the apron with an end step have been adapted to the prevailing tailwater conditions at the New Naga Hammadi Barrage site.

A key parameter in the hydraulic design of the sluiceway and downstream apron is the tailwater rating curve given in **Chapter 2.3**.

Due to the depth of the sluiceway sill, the outflow beneath the radial gates will be submerged for the entire range of discharges. The typical ogee form for spillway crests is, therefore, not applicable. In the case of very high floods, the gates would be fully lifted and the flow conditions would remain subcritical. As usual in such cases, the dimensions of the structure have been confirmed by hydraulic model tests.

The gates of the sluiceway structure are separated by 4 m wide piers. The block joint system of the civil structure of the sill provides for an expansion joint in the middle of each second opening, thus sectioning the structural system into 4 blocks. The two end blocks include each a dividing pier between gates and a reduced side pier which is integrated into the large sluiceway abutment and intermediate piers, but separated by expansion joints. On the powerplant side, the length of the sluiceway side blocks is 31.5 m adjacent to the intermediate pier and 33.5 m at the sluiceway abutment pier. The two central blocks have a length of 42.0 m each, and contain each two dividing piers of the sluiceway. With this block and joint system, only three of the gate openings contain a central expansion joint.

The block system was chosen with the expectation of sufficient density of the foundation soil at 42.0 m asl and with the positive experience at the New Esna Barrage where only negligible differential settlements have occurred. The relatively long blocks can bridge non-uniformities of the foundation parameters and keep possible dilatations between the blocks at a minimum. Details of the foundation conditions are described in **Chapter 7.6**.

A section along the axis of a sluiceway opening is shown on **Album No. 38**, extending over the entire range of the piers and the downstream apron with end step and concrete slab extension. The operating range of the gate and flap is restricted to the area between the runway beams of the main gantry cranes, including the upstream stoplogs. The slots for the downstream stoplogs are on the downstream side of the gantry crane runway beam.

There is a service bridge near the downstream stoplogs, and the space downstream on the piers accommodates the elevated road bridge with the ramp pillars, see **Album No. 38**.

Each of the gates with a clear width of 17.0 m and a height of 13.5 m, including the flaps, is moved by two oil-operated hydraulic hoists located in the piers. It is a safety requirement that the hydraulic hoists be capable of holding the gate with one servomotor only. The gates are capable of closing the sluices under action of gravity for any conditions of head and discharge.

All gates are equipped with an upper flap gate for discharging water in excess of the turbine discharge capacity (occurring in the months of June to August), for fine regulation of the headpond level and for flushing of floating weed should its removal not be possible. The flaps together with the turbines of the hydropower plant will enable release of the full river discharge during the high flow (high irrigation demand) season. In case of sudden load rejection by the powerplant, the sluiceway gates will automatically open to assume the discharge at which the powerplant was operating.

The stoplog elements are interchangeable with those from the navigation lock and are stored in the dogging device above each sluiceway opening. The elements can be raised and lowered by the main gantry crane under balanced head conditions only.

## 4.4.2    Navigation Locks

The two chamber navigation lock is constructed on the right bank of the river formed by El Dom Island. The following explanations refer to the layout and sections given in **Album Nos. 43 to 64**.

The approach on the upstream side of the navigation lock requires relatively large excavations and backfill with protective material up to 56.6 m asl. This allows passage of river traffic during construction when the headpond is still not impounded but the diversion canal is closed and the river flow passes the sluiceway. For this period of about 3 months, it is sufficient that only one of the two chambers can be used, e.g. that on the side of the river. The low upstream base of the approach to that navigation lock chamber is also required for enabling the river side navigation lock chamber to participate in evacuation of the emergency release from the HAD, as required by the MWRI for additional safety.

The GANT requires a minimum of 3.0 m depth of water in the navigation lock and above the downstream sill to make adequate allowance for future riverbed degradation. The maximum draught of ships navigating the River Nile is 1.8 m and a keel clearance of 0.5 m must be provided. For the minimum navigable river discharge of 350 m$^3$/s, the water level based on the tailwater rating curve is 57.9 m asl. Allowing for the anticipated tailwater decrease within 50 years (0.7 m), a water depth of 2.3 m would still remain. The large design ship (passenger ship) proposed by GANT as a standard for new construction, has a maximum draught of 1.3 m only.

For the period of construction of the closure dam, a total depth of water of 2.30 m is provided to allow for the maximum draught of 1.8 m plus 0.5 m keel clearance at a river discharge of 710 m$^3$/s. For the few weeks of the closure period when the river discharge is reduced to 350 m$^3$/s, the maximum permissible draught will be somewhat limited.

The double chamber navigation lock is a 228.8 m long structure with a double U-cross section (see **Album No. 50**), which structurally is composed of an U-shaped section for the riverside chamber, and an L-shaped section for the landside chamber. The riverside section integrates the separation pier between both navigation lock chambers, which consists in a hollow section backfilled with gravel. The expansion joint in the chamber bottom between the landside L-shaped section and the river side U-section will be compressed by the active earth pressure from the landside backfill. The joint is designed with a shear key to counteract uplift forces. The location and alignment of expansion joints and shear keys may be adjusted during construction design.

The complete dimensions of the navigation locks are shown on **Album Nos. 43, 44** and **50**. Perpendicular to the navigation lock chamber axes, there are expansion joints with shear keys which intersect the 10 structural blocks of each lock chamber.

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The most upstream block of the landside chamber includes a high level sill with the mitre gate, whereas the riverside chamber has a low level sill with a sector gate. The riverside navigation lock chamber is used as discharge and ship passage, against the end of construction when the closure dam is constructed and the headpond is still not impounded. It shall then later be able to participate in the evacuation of the emergency flood. For both purposes, the sector gate can be operated against streaming water. The upstream blocks of both navigation locks further contain the ship arresters and end markings of the chambers.

The following 5 blocks separated by expansion joints are of standard size and 22 m long each, and another 2 structural blocks thereafter contain the filling ducts for each of the two navigation lock chambers. The next structural block then contains the distribution or dissipation ports for filling and emptying, and the downstream mitre gates of the navigation locks. The most downstream structural block then contains the emptying ducts of both navigation locks, and carry the supports for the high level public road bridge.

To the left of the filling chamber blocks, there is a connecting structure between the navigation locks and sluiceway abutment pier. The connecting structure contains the two separate systems of ducts for filling and emptying each navigation lock. As can be seen on **Album Nos. 49** and **52**, the filling ducts intakes are located within the upstream bay formed by the connecting structure, whereas the outlets of the emptying ducts are located within the downstream part of the abutment pier of the sluiceway. The connecting structure contains one regulating gate for each operation of each duct, and one pair of stoplog slots at each of the regulating gates for maintenance. Within the navigation lock, both inlets to and outlets from the distribution/dissipation chambers are orientated in direction of the navigation lock axes.

The length and width of the upstream and downstream approaches to the navigation locks comply with the navigational requirements as required by GANT. The effective cross-current velocities measured in the Barrage Model at all navigable discharges both up- and downstream do not exceed 0.3 m/s. In the vicinity of the barrage, higher cross-current velocities are prevented by guide walls. In the upstream area, this results in massive structures due to the low level of the approach.

Filling and emptying times of the navigation lock chambers were calculated and tested in the hydraulic model, not exceeding 10 minutes. Hydraulic model tests revealed that these times are not critical for the hydraulic forces acting on ships inside the navigation lock chambers.

### 4.4.3 Hydropower Plant

#### (1) Powerhouse Design

As shown in **Album No. 28**, the powerhouse consists of 4 unit bays, each two being combined into one structural unit block of the civil structure. The overall length of the unit blocks is determined by adding the dimensions of the structures in which the following single components (see **Album No. 29**) are contained:

(i) trash rack with raking machine,

(ii) upstream stoplogs for dewatering of the turbine pit,

(iii) generator-turbine bulb unit with foundation, vertical shaft bearing with integrated access and cable shafts, and subsequent turbine encasing with distributor blades and turbine runner, and

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(iv) draft tube with the transition of the discharge section to the outlet square shape, and tailwater gates at the end of the daft tube.

The total length required for all standard components which determine the total length of the powerhouse unit blocks, is 72.0 m. This block length may be intersected by an expansion joint behind the turbine runner section. The 33.9 m width of a two unit block results from the required intake areas and the pier widths. The shape of the pier and intake was tested in the hydraulic model, to avoid the formation of vortices. It is hydraulically important that the trashrack panels totally cover the separation piers between the units.

The powerplant design contains a main powerhouse hall, which extends over the total length of the generator-turbine sets including the central portion of the intermediate and abutment piers, and which is closed against the exterior by removable roof covers above the units and both piers. The main powerhouse hall is served by an interior crane for a maximum load of 10 t, sufficient for the loads to be moved for maintenance of mechanical and electrical equipment. For extraordinary cases of repair requiring larger load capacity, the roof cover above single generating units can be removed and the main gantry cranes be used.

The public traffic road is well separated from all operations concerning the powerhouse, by the service road, continuing from the service bridge on the sluiceway. The service road covers a width of about 6 m downstream from the runway of the main gantry cranes. The space below both service and public traffic roads and above the draft tubes is used to accommodate all electrical and auxiliary equipment, except the main transformers which are in the outdoor switchyard on the west bank. Substantial space in both side piers is integrated into the powerhouse compartmentalisation.

The outline design of the civil structure is shown in sections on **Album Nos. 29, 35** and **36** and in plan for different levels on **Album Nos. 28** and **30** to **34**. The overall dimensions of one structural block with two units is 33.9 m in width and 72.0 m in length, with a total height between lowest foundation level and road platform of 30.9 m. The expansion joints of the blocks are located in the central separation pier, and at the side of the abutment and intermediate piers.

The principles on which the design is developed are:

(i) In order to obtain sufficient trashrack area at limited depth and width, the trashrack is inclined. The gross horizontal velocity at the design discharge of 1,280 m³/s is 0.9 m/s. The trashrack area cannot be closed upstream. Two trashrack cleaning machines are provided to remove trash and waterweeds arriving from upstream. Trash will be accumulated in a transverse ditch above the intake and be removed.

(ii) The area of each generator bulb and turbine is closed against the flow, and is accessible through the main powerhouse hall which covers the area above the steel-lined components of the turbine canal and draft tube. The 6.8 m diameter bulb and turbine are only vertically supported within the arrangement of a large steel-lined column with two internal shafts, the generator and the cable shaft. Within this supporting structure, there is permanent access to the interior of the bulb generator. The cable shaft within the supporting structure contains all cables for connection to the medium-voltage switchgear.

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(iii) The turbine shafts provide access by loads to the turbine floor, to the wicket gate motors and the drainage and dewatering gallery. The gallery connects all unit blocks with the dewatering sump in the left abutment pier, which is accessible by a staircase and elevator.

Access underneath the steel casing of the turbine runner and distributor blades is from the turbine floor. For small scale access to the turbine canal, there is a manhole from the turbine floor. Large scale access through the turbine shaft into the turbine canal is only possible when the upstream and downstream emergency gates are closed and the water from the turbine canal has been evacuated. For access, the cover of the steel casing has to be dismantled.

(iv) Dewatering of the turbine canal and associated draft tube is possible by lowering the bulkhead gates upstream in front of the generator bulb and downstream at the end of the draft tube. The upstream gates consists in two bulkhead panels and can be closed under flow conditions which could occur when the turbine gates close whilst being damaged. The gate panels are stored in the upper part of each gate slot and can be lowered by the main gantry crane. The downstream stoplogs are set for erection, inspection and maintenance of the units by a crane from a separate service bridge. Stoplogs are stored above the draft tube outlets. The turbine canal and draft tube can be totally dewatered into the dewatering gallery which discharges into the pump sump in the intermediate pier between powerplant and sluiceway.

(v) Cable shafts are routed within the end piers and from there connected by galleries to the cable floor directly on top of the draft tubes. All electrical equipment is contained in the rooms above the cable floor, accessible from the main powerhouse hall and the staircases in each end pier. Separate compartments are provided in each unit block near the pair of units, for the 11 kV cubicles and auxiliary transformers, the excitation and voltage-regulating cubicles, auxiliary power supply, and low voltage distribution boards. Other equipment common for all units, and stores, are contained in the compartments on the intermediate floor below the road platform. The cable duct from the abutment pier leading to the conventional switchyard is indicated in **Album No. 28**.

The location, and general arrangement of the switchgear are outlined on **Album No. 6, 124 and 125**.

The 220 kV switchyard of 130 by 100 metres in size will be located on the west bank adjacent to the backfilled diversion canal some 400 metres from the powerhouse at 69.00 m asl. The switchyard will be equipped with an outdoor, double busbar configuration, low-rise layout type switchgear that includes five bays plus a spare area for two future bays:

- two 11 kV - 11 kV / 220 kV transformer bays fed by the four generators in the powerhouse through 11 kV cables;

- two feeder bays supplying the Double-Circuit Over Head Transmission Line (DC-OHTL), 32 km long, connected to the existing Naga Hammadi substation;

- one busbar coupling bay;

- provision for two future feeder bays non-equipped except the busbars and their supporting gantry.

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The line connects to the Unified Power System (UPS) via the Naga Hammadi substation located approximately 26 km from the New Barrage, which includes all voltage levels from 11 kV to 500 kV.

## (2)  Operating Conditions

All river discharges will normally pass through the hydropower plant except during the high flow period during the months of June to August. With the units operating, the maximum discharge capacity of the hydropower plant will be 1,670 m³/s. Maximum generating capacity of the generators is 64 MW (4 x 16 MW) at prevailing head conditions, which cover a net head range of 3.80 m to 8.00 m. The highest head of 8.75 corresponds to a headpond level of 65.90 m asl and a tailwater level of 57.86 m asl (based on a discharge of 350 m³/s) after 50 years of projected riverbed degradation. The lowest head corresponds to maximum normal river flow during the high-flow season of 2,530 m³/s. During floods, the hydropower plant will operate in combination with the sluiceway up to a maximum total river discharge of 3,340 m³/s, which corresponds to a minimum head of 2.8 m. For higher river discharges, the powerhouse would be closed and would not be able to participate in flood release.

The most critical case for turbine operation is the full-load rejection, e.g. by transmission loss or similar failure of the powerplant. If this occurred the distributor of the turbines would have to close, which would result in a high-speed rise of the turbine runner and surges in the headpond and the tailwater of the New Barrage. The sailing operation of the units is therefore applied, which limits the overspeed and maintains about 50% of the discharge through the powerhouse for some 3 minutes. During this time, the sluiceway gates would be activated and within the following 3 minutes opened to convey the former powerplant discharge. Details of this operation are given in **Volume 2, Chapter 2.5.7**.

The dimensioning of the generating units is described in **Volume 2, Chapter 2.5**. The low setting of the axis of the bulb turbines at 51.1 m asl, that is 6.25 m below minimum tailwater level, takes into consideration with safety the reduced tailwater levels which are estimated to establish within 50 years by riverbed degradation.

### 4.4.4  Diversion Closure Embankment and Right Bank Closure Dyke

The Diversion Closure Embankment is constructed when the sluiceway structure is operable and the cofferdams are removed to pass the river flow through the sluiceway before the period of wet testing the generating units. The downstream part of the closure embankment continues in the axis of the future public road on the main concrete structures across the 200 m wide (135 m at base) diversion canal. After completion of river closure, the area of the diversion canal upstream of the closure dam will be backfilled to constitute the area for establishment of administration buildings and workshops.

The embankment section shown on **Album No. 21** comprises upstream and downstream rockfill toes. The proposed scheme to construct the downstream rockfill toe is given in **Album No. 21**, and has been based on hydraulic model investigations in the Barrage Model. The rockfill on the upstream side of the embankment (headwater side) has not to be dumped in streaming water and will be of small grain size to prevent a permanent flushing of sand into the rockfill. Under the protection of the two rockfill embankments, the closure embankment will then be completed by end-dumping sand from the left-bank deposit. The outer slopes of the rockfill toes will be at 1V:3H and dumped to elevation 62.0 m asl to allow subsequent works to be completed under dry conditions for river discharges up to some 2,500 m³/s (high irrigation demand season).

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Generally, no compaction under water will be performed during construction. When the dumping height is above the water level, a dynamic roller will be used, resulting in very small expected long-term settlements. A geotextile and finally riprap protection will preserve the slopes of the closure dam against erosion of sand fill in the range in which water levels can vary.

As for the cofferdams, the slope stability of the closure embankment will be well above the required factor of safety against slope failure of 1.3. For potential slip surfaces passing through both sand and rockfill, the location of the more shear-resistant rockfill at the toe of such slip surfaces and the berm width of 7.5 m will favourably influence the slope stability. Detailed calculations of slope stability are given in **Chapter 7.2.6**.

The embankment has to be sealed by a diaphragm wall near its downstream face. Seepage calculations have confirmed the approximate location as shown on **Album Nos. 21** and **70**, and approximate depth to level 39.0 m asl. It will be a slurry trench wall, similar to that underneath the main concrete structures, with a width of 0.8 m. The top of the wall is at 66.0 m asl.

The seepage exit gradient at the downstream toe of the embankment is acceptable, being less than the critical gradient for fine and medium sand.

The protection dykes on El Dom Island and the right bank closure dyke in the old river flood channel east of the island are shown on **Album No. 21**. A permanent impoundment will exist only in front of the backfill. No diaphragm wall is required, as the natural seepage through the backfill of the floodway will be acceptably low. The closure embankment will be protected by a geotextile covering the waterside of the embankment and the toe. River slope protection will cover the geotextile.

## 4.4.5 Seepage Cut-off Measures

The foundation levels of the powerhouse at 38.0 m asl and of the sluiceway at 41.90 m asl could be within a layer of gravel and cobbles shown on the Geological Sections, **Album Nos. 9.5, 9.6, 9.7, 9.11** and **9.12**. This layer, with a permeability of $2 \cdot 10^{-3}$ m/s derived from evaluating pumping test, is decisive for seepage and uplift. Hence, a seepage cut-off wall is arranged below the upstream ends of the sluiceway and powerhouse unit blocks to intersect that layer and to penetrate into the clay layer, which has an upper boundary between elevations 13 and 16 m asl.

The diaphragm wall will be a slurry trench cut-off wall as for the temporary works. The thickness will be 0.8 m.

To ensure that the area of the portal blocks of the navigation locks with the dissipation chamber are safe against uplift for the case of emptying during maintenance, the diaphragm wall will continue from the sluiceway to the navigation lock upstream of the filling blocks, as shown on **Album Drawing No. 71**. Beyond the navigation lock on the right bank, it will be connected to the diaphragm wall of the temporary works for the construction pit in order to considerably lengthen the seepage path around the navigation lock.

The diaphragm wall on the left side will continue under the abutment pier and then within the closure embankment. The depth of the diaphragm wall - when outside of the powerhouse abutment - will decrease with distance from the concrete structures and does not need to connect to the clay layer. This has been confirmed by seepage calculations. The permanent diaphragm wall will intersect the temporary sealing element around the construction pit.

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### 4.4.6   Erosion Protection of the Riverbed and Approaches to the Structures

It is a requirement for guaranteeing the long-term stability of the permanent concrete works, that the riprap protection within the area surrounded by the diaphragm wall of the construction pit plus an additional coverage of about 150 meter downstream from the wall should be absolutely stable for all admissible discharge operation modes, up to the evacuation of the emergency flood (7,000 m$^3$/s). The adjacent river bottom reaches upstream and downstream shall be safely protected for release of the 100 years flood discharge, i.e. 2,900 m$^3$/s, which has never happened since the implementation of the HAD.

Riverbed protection types have been defined to achieve a sustainable protection of the bed and the banks of the river. Failure mechanisms of riprap are related either to erosion of individual particles from the surface or from washing out of underlaying filter material and/or subsoil. In general the riverbed protection (types I to VI) is made up of:

- a top layer of riprap (types R1 to R6, see **Chapter 5.8**) characterised by its mean diameter $D_{50}$, and weight $W_{50}$,

- one or two transition layers of smaller grain sizes below, or even no transition layer, and

- a geotextile filter on top of the original riverbed.

At the barrage site, the riverbed material of the Nile is characterised by a mean diameter $D_{50}$ = 0.23 mm and an uniformity coefficient of $U = d_{60}/d_{10} = 2.1$. At the slopes even silt may be encountered. In order to achieve an effective filter or transition design for such fine material, geotextiles are used for the lowest layer on top of the sand at all permanent structures. On geotextile, a gravel layer is placed for its protection against sharp edges of larger material when sunk.

The riprap riverbed shall extend over the full width of the river channel. At the transition from the riverbed protection to the natural bed, scouring is most likely to occur. These transition zones are designed to prevent propagation of scour towards the diaphragm wall remaining from the construction pit, see **Album No. 26**.

Six sizes of riprap material with specific density of 2,200 kg/m$^3$ were defined for the purpose of riverbed and bank protection and their distribution determined by hydraulic model tests to achieve stability, see **Album No. 24**. The riprap types and classes are defined by weight. The design of riverbed protection shown on the drawings is related to the Esaweia limestone material, which was used in the model tests.

Type I, the heaviest riprap with $D_{50}$ = 0.75 m or $W_{50}$ = 620 kg is placed in the area which is subject to frequent and continuos high flow velocities, i.e. immediately downstream of the powerhouse exit. In case that the riverbed protection is arranged by placement of single stones of rather uniform size then riprap stone size type R2 can be applied instead.

Type II, with $D_{50}$ = 0.57 m or $W_{50}$ = 315 kg is applied downstream over some 125 m in flow direction, and cover a width of 270 m in the areas of exit flow from sluiceway and powerhouse. Additionally, the navigation lock approaches are provided with the same type of riprap. Riverbed protection type II is also applied as toe protection along the intermediate pier between powerhouse and sluiceway and in front of the right abutment pier of the sluiceway.

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Type III, with $D_{50} = 0.39$ m or $W_{50} = 100$ kg will be placed in the areas downstream and adjacent to type II, covering areas of some 250 m length of the left bank downstream of the powerhouse, and a riverbed area of some 125 m length downstream of powerhouse and sluiceway. It covers also the berthing areas over a width of some 100 m and a length of 400 m, including the slopes on the right bank.

Upstream, riverbed protection type III will extend over some 130 m length and over the entire width of sluiceway and powerhouse. This includes also corresponding parts of the left bank slope and of the navigation lock approach.

Type IV with $D_{50} = 0.26$ m or $W_{50} = 30$ kg is foreseen downstream and adjacent to the type III for some 150 m over the entire width of the river and on the remaining right bank slope. The downstream face of the diversion closure embankment shall also be of the type IV, designed against the attack by wind induced waves. There remain also some areas in the headpond for protection by type IV.

Type V, with $D_{50} = 0.17$ m or $W_{50} = 8.5$ kg is placed as riverbed protection over a larger distance in the headpond where maximum stream velocities during the emergency release are small, and on slopes of the diversion canal.

Type VI, with $D_{50} = 0.10$ m shall only be used for temporary works, in the diversion canal bottom and on river slopes at distance from the barrage structures.

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# CHAPTER 5

# HYDRAULIC DESIGN

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## 5.    HYDRAULIC DESIGN

### 5.1    General

Present and future water level conditions at the unimpounded river near the new Barrage site were calculated by application of measured water levels downstream of the old Barrage and applying a water surface calculation programme (HEC-2). The results of the latter are given in **Section 2.3** of this design report.

In view of the many details of river flow phenomena which could not be mathematically modelled, 3 physical models were established in the Hydraulic Research Institute (HRI) in Cairo, for the following purpose:

### (1)  Sluiceway Detail Flume Model

This model of one gate opening between pier centrelines, at scale 1:21, was used for:

- investigation of subcritical flow conditions at fully open gates with the particular objective to determine the headpond level rise;

- investigation of supercritical flow conditions at normal operation with partially open gates to optimise the shape and length of the downstream apron;

- verification of riprap stability by means of measuring bottom velocities and velocity fluctuations at pre-determined distances downstream of the apron;

### (2)  Barrage Model

This model of the entire barrage works between 0.8 km upstream and 1.2 km downstream, at scale 1:30, was used for:

- confirmation of the general arrangement of the layout of the components and assessment of navigation conditions in the approach to the navigation lock;

- optimisation of flow conditions in the immediate vicinity of the structures including upstream the shape of piers, guidewalls, power intakes etc. and downstream the shape of the sluiceway apron with adjacent riverbed protection;

- shaping of the diversion works regarding their hydraulic performance including navigation conditions and riprap stability;

- performance of the river closure;

- confirmation of the ability of the navigation lock to participate in the evacuation of the emergency flood;

- establishing gate operating rules for the sluiceway gates.

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### (3) Navigation Lock Model

This model for one navigation lock chamber including filling/emptying canals and chamber, at scale 1:20, was used for:

- optimisation of the filling and emptying system of the two navigation locks;
- determination of the forces acting on a ship during emptying and filling of the lock chamber;

Most of the investigations were carried out in the post - feasibility phase and before and during the tender design. Additional tests in the Barrage and the Navigation Lock model were then carried out at the end of the tender design.

The results of the hydraulic model tests are available in 3 volumes, each volume covering the tests with one of the models. The reports on the Barrage Model and the Navigation Lock Model reports were amended by additional investigation appendices. With availability of the results from hydraulic model testing, the hydraulic dimensioning and design was improved against the initial hydraulic dimensioning with standard calculation methods.

## 5.2    Sluiceway Hydraulics

### 5.2.1    Sluiceway - Discharge Conditions

Typical flow conditions through the sluiceway are:

**(1)    0 - 2,400 m³/s**

The powerhouse normally discharges all the river discharge up to a maximum plant capacity of 1,670 m³/s. The remainder of the river discharge is released by the flaps on the sluiceway gates. The radial gates are only lifted when the power plant stops operation.

**(2)    0 - 5,700 m3/s approx.**

Partially open gates with supercritical flow passing underneath the gates, forming a fully submerged jet in the tailwater.

**(3)    Above 5,700 m3/s approx.**

Sub-critical flow conditions with fully open gates. The headpond water level rises above normal operating level.

### 5.2.2    Sluiceway - Discharge Capacity

The sill shape and level of 52.80 m asl was confirmed by an optimisation process in the Sluiceway Detail Model, together with the adjacent apron. The objective of the investigations was to maintain the headpond water level rise within the limitation of maximum historic flood marks from the period before implementation of the HAD (e.g. 67.40 m asl in 1959 and 1960).

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Results of hydraulic model tests differ by the tailwater conditions which are prevailing now and which may have changed by downstream riverbed degradation within 50 years. The formulae of the two tailwater curves are given in **Section 2.3** of this design report.

Results from hydraulic model tests for submerged outflow under the gates are given in **Figure 5.1** for the normal case that all gates are equally operated. The headpond levels which establish with the fully withdrawn gates have been measured by hydraulic model tests, and are summarised in **Table 5.1**.
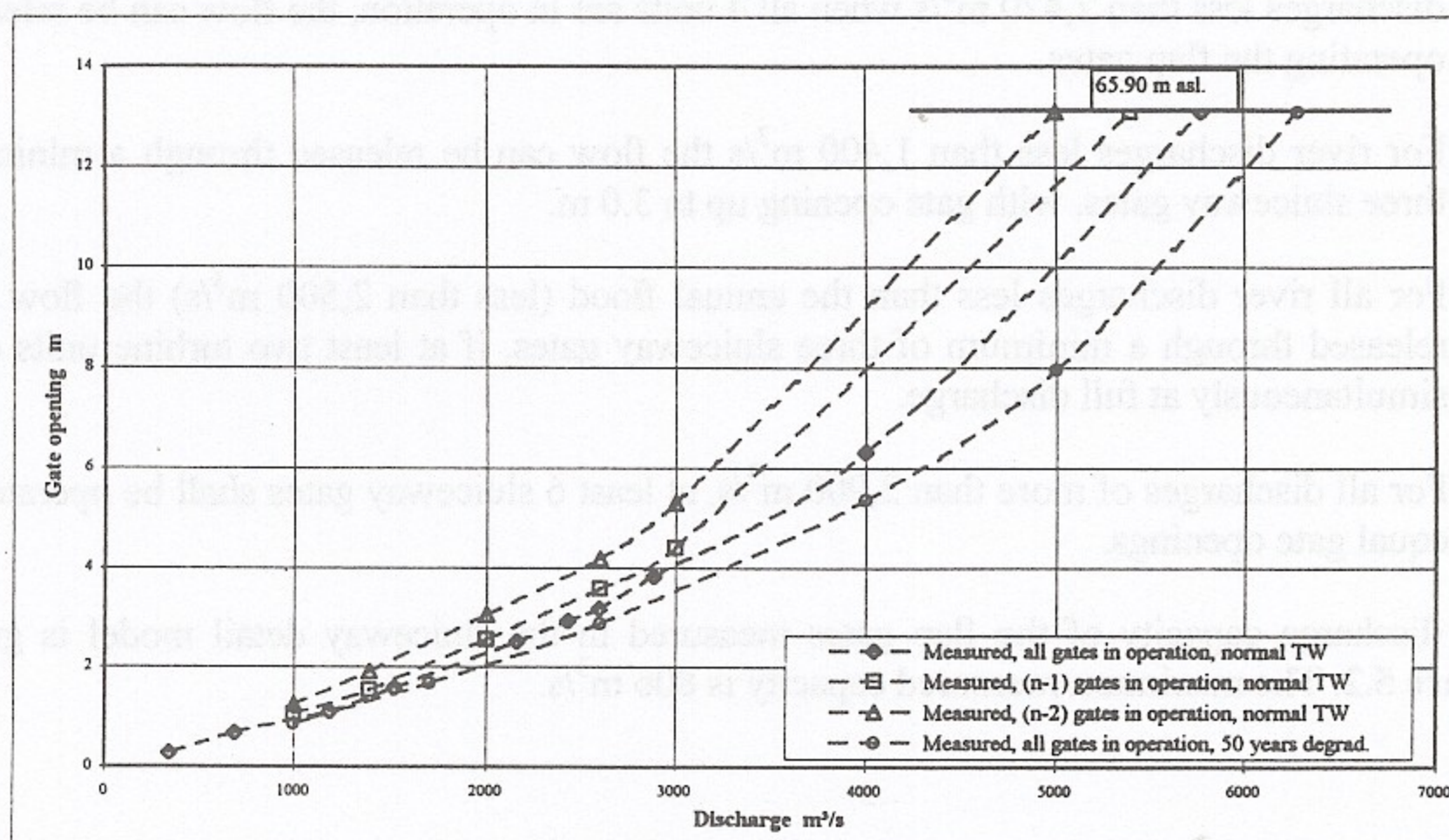


Figure 5.1: Discharge Capacity of Sluiceway

Table 5.1: Barrage Works - Flood Evacuation Capacity

| Discharge Waterways | Discharge $m^3/s$ | Headpond Level m asl | Tailwater Conditions m asl | Water Level Difference m |
|---|---|---|---|---|
| 7 gate openings | 7,000 | 67.05 | prevailing 66.46 | 0.59 |
| 7 gate openings plus 1 navigation lock chamber | 7,000 | 66.88 | prevailing 66.46 | 0.42 |
| 6 gate openings (one gate not available) | 7,000 | 67.24 | prevailing 66.46 | 0.78 |
| 7 gate openings | 5,800 | 65.90 | prevailing 65.46 | 0.44 |
| 7 gate openings | 6,200 | 65.90 | after degradation 65.27 | 0.63 |

For the case that only the sluiceway is operated but not the hydropower plant, there is measurable difference between the capacity of the sluiceway bays. For a total river discharge up to 2,400 $m^3/s$ discharges at gates nos. 1 to 3 (from left to right) are higher than average, having a maximum at gate no. 1. The lowest discharge capacity has gate no. 7 (8 % less than average).

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There is a significant decrease of the discharge capacity of gate no. 1 with the emergency release, i.e. 10 % less compared to the average of the sluiceway.

Tests of accidental full opening of only one gate showed deep scouring downstream of the concrete apron. Restrictions to operate the sluiceway gates are imposed through the rule not to operate groups less than 3 gates. The permissible difference in gate openings by groups was determined by means of hydraulic model tests with the following results:

a) For river discharges less than 800 m³/s without powerhouse operation, and at river discharges less than 2,470 m³/s when all 4 units are in operation, the flow can be released by operating the flap gates.

b) For river discharges less than 1,400 m³/s the flow can be released through a minimum of three sluiceway gates, with gate opening up to 3.0 m.

c) For all river discharges less than the annual flood (less than 2,500 m³/s) the flow can be released through a minimum of three sluiceway gates, if at least two turbine units operate simultaneously at full discharge.

d) For all discharges of more than 2,900 m³/s, at least 6 sluiceway gates shall be operated with equal gate openings.

The discharge capacity of the flap gates measured in the sluiceway detail model is given in **Figure 5.2**. The maximum measured capacity is 806 m³/s.
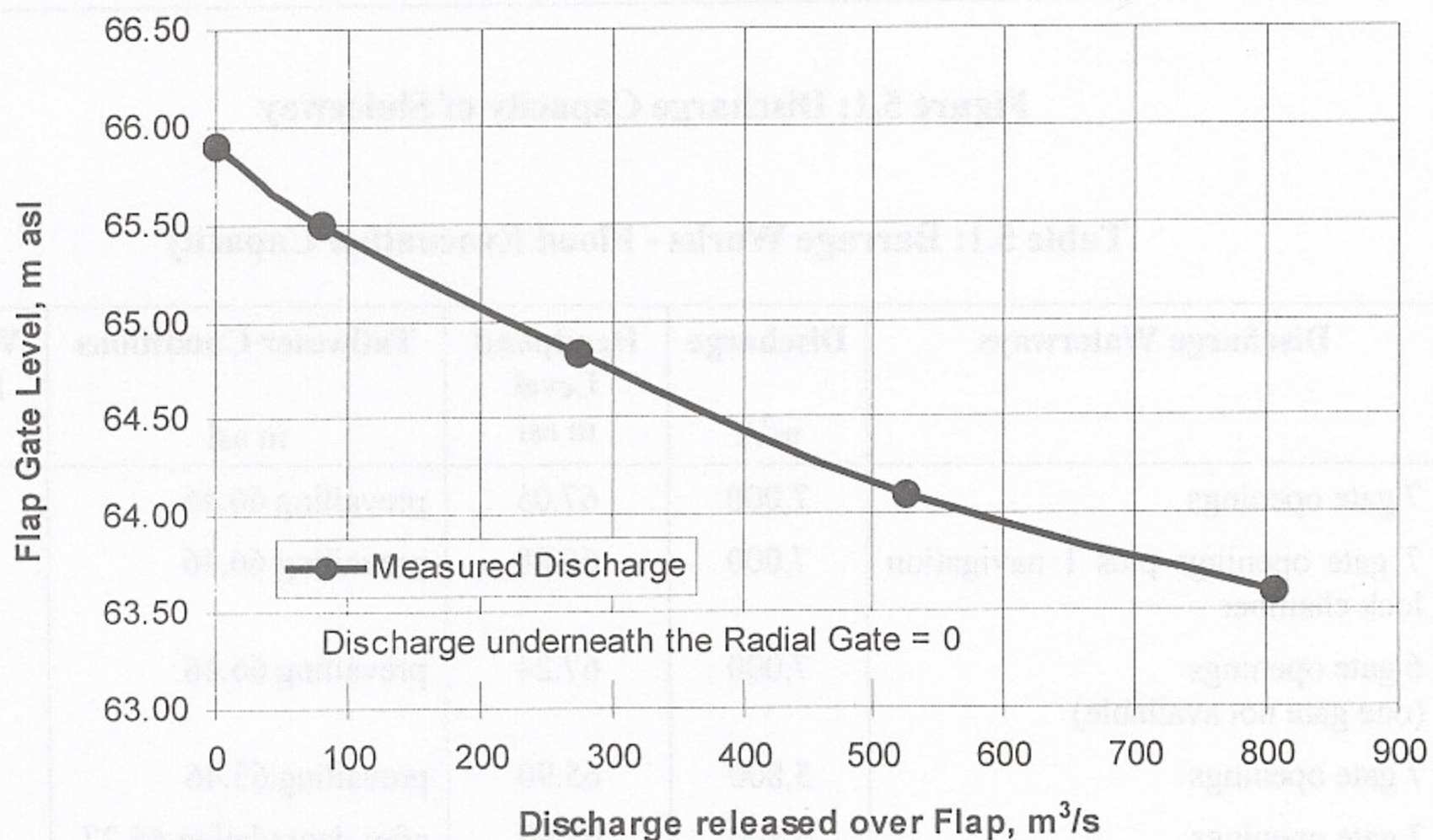


**Figure 5.2: Discharge Capacity of Flap Gates**

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### 5.2.3   Sluiceway Apron

Tests in the 2-dimensional Sluiceway Detail Model indicate that the critical range of Froude Numbers establishes for operation at partial gate opening in the range from 2,000 to 4,000 m³/s. For a discharge of 2,600 m³/s, intensive generation of waves in the downstream reach was observed, indicative for the oscillating hydraulic jump at Froude numbers of about Fr = 2.1.

Best performance of the apron was achieved for a 30° inclined backslope of the sill, the stilling basin floor at 46.00 m asl and a vertical step at the end of the apron leading to a platform on elevation 48.00 m asl. These dimensions and shape were included in the design. On the platform downstream of the vertical end step, a 30 m long horizontal concrete slab is provided to avoid scouring. The riverbed protection continues by adjacent riprap protection starting with a 20 m long horizontal reach, followed by a 1: 14 slope, both covered by riprap type R2.

The adopted design of the sluiceway sill and apron is shown on **Album No. 38**.

### 5.3   Powerhouse Hydraulics

### 5.3.1   Approach Flow Conditions

Approach flow conditions to the powerhouse have been tested in the Barrage Model. The approach flow towards the powerhouse was found to be slightly distorted, so that modifications at the intermediate and left bank piers were necessary to achieve an almost uniform approach flow to each of the 4 units.

The asymmetrical shape of the intermediate pier in combination with an increase of its length against the original design showed the best results. The bottom configuration, i.e. the arrangement of the transition from the powerhouse approach canal bed and the sluiceway approach canal bed, turned out to have minor effects on the approach flow to the powerhouse. In order to avoid scouring in case of occurrence of the design flood a mild sloped transition 1: 5 was built in.

Modification of the left bank abutment pier became necessary to eliminate a dead water zone on the left bank which caused reduced flow to intake no. 1 and intensified vortex formation. The situation improved by reducing the length of the pier in the headwater, thereby extending the forebay area into the left bank.

The modifications of the intermediate pier and the left abutment pier improved the approach flow so far that at intake no. 1 the discharge was 6 % above average and at intake no. 3 the discharge was 3.3 % below average for the maximum turbine discharge of 1,670 m³/s.

Approach flow velocities were measured (as in the feasibility study tests) in three vertical sections 55 m upstream of the project centre line of each unit bay. The verticals are located in the unit bay centre and 3.45 m to the left and right. Measurements of 2-D velocity components were conducted at 3 points, at 20%, 50%, and 80% of the depth of flow. The velocities vary between 0.80 and 1.07 m/s over all four intakes.

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### 5.3.2 Head Losses

The detailed head loss computation under consideration of trashrack and turbines resulted in the following formula, see **Section 11, 5.4** of **Volume 2** - Tender Design Report:

$$\Delta h = 6 \cdot 10^{-7} \, Q_T^2, \text{ in m}$$

### 5.3.3 Flow Separation and Vortex Formation at the Intakes

For the original layout at the powerhouse bay piers flow separation occurred which in turn was supposed to intensify the tendency to vortex formation. Therefore, the shape of the piers was modified. In this conjunction the arrangement of the trashrack was adapted to the new design of the piers and they were placed at the upstream face of the powerhouse bay piers.

In the course of the hydraulic model tests the maximum powerhouse design discharge was reduced due to detailed studies of the turbine characteristics from 1,842 m³/s to 1,670 m³/s.

Testing of the original layout of the powerhouse showed vortex formation according to Type 2 to 3 (**Figure 5.3**) for a simulated powerhouse discharge of 1,840 m³/s. Vortex formation can be attributed to formation of a stagnation surface water zone directly in front of the intakes. The above mentioned measures in conjunction with the modification of the contour of the intake soffit were applied as successful remedy measures and:

- re-shaping of one left abutment pier,
- re-shaping of the intermediate pier with an elliptic pier face,
- re-location of the pier face towards the intake section, and
- optimisation of the contour of the intake soffit with the aim to reduce the extent of the stagnation zone near the surface or even to remove it.

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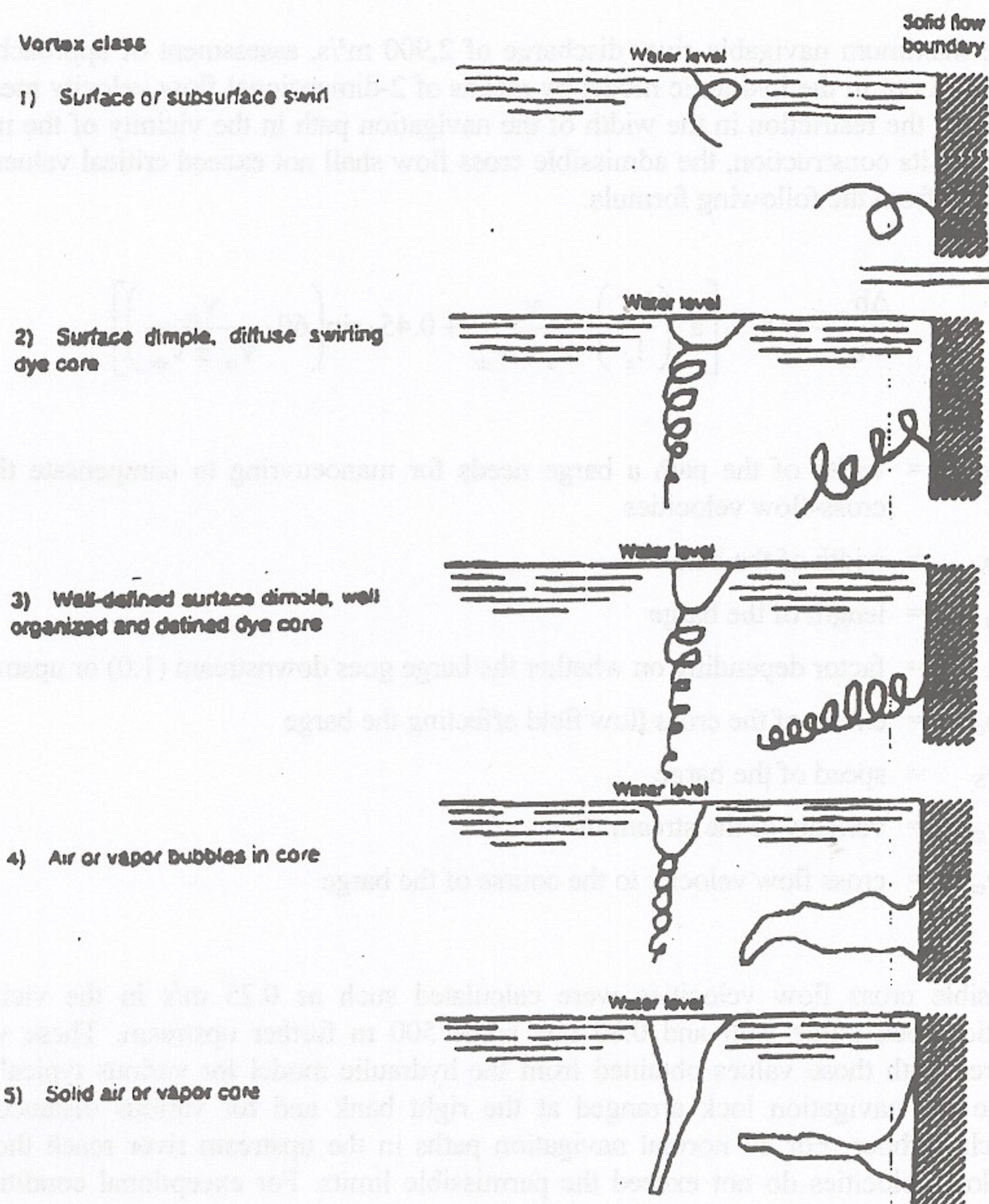


**Figure 5.3: Vortex Classification System - Japanese Recommendations**

### 5.3.4 Simultaneous Operation of Powerhouse and Sluiceway

From earlier tests it was found that approach flow conditions were critical for the combination of 1,700 m³/s passing the powerhouse and 1,200 m³/s passing the partially lifted sluiceway gates. The considered operation mode corresponds to a flood having a return period of 100 years.

Since simultaneous operation of the powerhouse and the sluiceway happens for less than 3 months per year, optimisation of the pier shape was performed giving priority to powerhouse operation. However, the simultaneous operation shows acceptable flow conditions at the intermediate pier also.

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## 5.4 Navigation Conditions and Navigation Lock

### 5.4.1 Approach Conditions

For the maximum navigable river discharge of 2,900 m³/s, assessment of approach conditions was carried out in the hydraulic model by means of 2-dimensional flow velocity measurements. Because of the restriction in the width of the navigation path in the vicinity of the new Barrage and during its construction, the admissible cross flow shall not exceed critical values which can be derived from the following formula.

$$\frac{\Delta b_{m,cr}}{b_s} = 8.5 \cdot \left[ a \cdot \left( \frac{b_{cr}}{l_s} \right) \cdot \frac{v_{cr}}{v_S \pm v_{str}} + 0.45 \cdot \sin\left( 60 \cdot \frac{v_{cr}}{v_{cr} \pm v_{str}} \right) \right]$$

where

$b_{m,cr}$ = width of the path a barge needs for manoeuvring to compensate the effect of cross-flow velocities

$b_s$ = width of the barge

$l_s$ = length of the barge

$a$ = factor depending on whether the barge goes downstream (1.0) or upstream (0.67)

$b_{cr}$ = extent of the cross flow field affecting the barge

$v_S$ = speed of the barge

$v_{str}$ = velocity of the stream flow

$v_{cr}$ = cross flow velocity to the course of the barge

Permissible cross flow velocities were calculated such as 0.25 m/s in the vicinity of the navigation lock guide wall and 0.80 m/s some 500 m further upstream. These values were compared with those values obtained from the hydraulic model for various typical navigation paths to the navigation lock arranged at the right bank and for various distances from the approach harbour. For all normal navigation paths in the upstream river reach the prevailing cross flow velocities do not exceed the permissible limits. For exceptional conditions, i.e. an accidentally taken unusual navigation path and operation of the sluiceway only at high river discharges, the cross flow velocities are higher than the permissible values. These exceptional conditions were not applied as design conditions for determining the location of the navigation lock. For all normal navigation paths measured cross flow velocities are acceptable under all investigated conditions in both upstream and downstream reaches.

### 5.4.2 Length of the Upstream and Downstream Navigation Lock Guide Wall

The length of the upstream guide wall to the navigation lock was varied in the hydraulic model with the aim to shorten the wall as much as possible. Cross flow velocities were measured at the upstream end of the wall for the maximum navigation discharge of 2,900 m³/s when passing through powerhouse and sluiceway. The calculated permissible cross-flow velocity at the upstream end of the guide wall is 0.25 m/s. The recorded mainstream velocities at that location are 0.30 - 0.35 m/s, but the maximum cross-flow velocities are only 0.13 m/s.

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It was further tested if openings in the guide wall could have an effect on ships entering the navigation lock. Although this was not the case, a perforated guidewall could be designed to save construction quantities.

### 5.4.3 Participation in Flood Evacuation

The required ability of the riverside navigation lock chamber to participate in evacuation of extreme floods needs a low level approach channel and inlet sill, e.g. at 56.60 m asl. It further requires a gate type at the upstream head which can be opened and closed against the high upstream velocities which could occur during release of floods through the navigation lock chamber.

Test series showed additional release through the navigation lock reduces the headpond level slightly. For the design flood of 5,700 m³/s and prevailing tailwater conditions the lowering of the headpond level is 6 and 7 cm, respectively, if all 7 gates and only 6 gates are open.

For of the emergency flood of 7,000 m³/s the lowering of the headpond level is more significant with 17 and 22 cm, respectively.

Although navigation lock and sluiceway bays have the same width, the discharge capacity of the navigation lock is less. This is due to the higher sill elevation (56.60 m asl) compared to the sluiceway crest (52.80 m asl). The rise of headpond level when substituting one sluiceway opening by the navigation lock is between 6 and 21 cm for different cases.

At all conditions considered, the headpond level remains below the pre-HAD flood marks of 67.40 m asl (even if two sluiceway gates are not available).

From tests with tailwater levels after 50 years of riverbed degradation, the headpond levels during extreme flood events resulted substantially lower, i.e. between 32 and 54 cm.

### 5.4.4 Filling and Emptying

The lock filling and emptying system consists of separate rectangular concrete ducts for each of the navigation lock chambers. The filling ducts are supplied from the headpond intakes in the connecting structure between the downstream navigation lock heads and the sluiceway abutment pier. The emptying ducts return from the filling ports through the connecting structure outlets in the area of the sluiceway apron. The duct systems from their intakes and outlets are orientated perpendicular to the corresponding navigation lock chamber axes, and from the axis they continue in axial direction.

At only a short distance and with a transition, the ducts join the lower levels of the distribution/ dissipation ports of each navigation lock. There is a separation wall throughout the duct bends and the distribution ports, which is needed to maintain equal emptying/filling discharges on each side of the ports. The need for the continuous separation wall is a result from the hydraulic model tests.

A requirement for the systems was that total filling or emptying times of the navigation lock chamber at highest differential head should not exceed 10 minutes. The water velocities in the concrete ducts shall not exceed 6 m/s, and the gates shall not be exposed to cavitation.

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The function of the dissipation/distribution port was tested in the hydraulic model for both navigation lock duct systems, and was iteratively improved by variation of size and distribution of the rectangular perforations in the lower deck and of the spacing and shaping of beams in the upper deck flash with the bottom of the navigation lock chamber. The function of the lower perforated deck, see **Album No. 48**, has to be seen in the distribution of both, filling and emptying discharges over the surface area of the part. For emptying, the submergence of this lower deck by the chamber water surface is relevant to avoid formation of vortices which could disturb the emptying flow and cause cavitation. The upper deck is slotted, and the beams between slots are formed to achieve an adjustment of the direction and concentration of filling and emptying discharges, and need a special shaping and spacing of the beams as obtained from hydraulic model tests, see **Album No. 52**.

Hydraulic model testing resulted in satisfactory conditions regarding the filling time and the hawser-stresses on the "design ship" incurring during filling and emptying cycles. The maximum resulting longitudinal force on the design ship was 22 kN during filling and 14.6 kN during emptying. During the initial period of 4 minutes of filling/emptying, the duct gates shall linearly open/close. This is a condition imposed by minimising the hawser stresses within the admissible filling time of ten minutes.

Hydraulic model tests further revealed that there is no significant impact of transversal forces on ships moored next to the dissipation port. Unfavourable flow phenomena such as bulk turbulence and vortex formation above the filling port are so far avoided that they do not affect ships in the chamber. The arrangement of the dissipation chamber close to the downstream gate causes a slope and translatory waves in the navigation lock chamber during filling and emptying. Therefore, ships have to be moored in the navigation lock chamber during sluicing.

### 5.4.5 Dimensions of the Navigation Lock Chamber

The General Authority of Nile Transport (GANT) had established design ship dimensions for planning of the navigation lock chamber dimensions. This is a cruise ship with a draught of 1.80 m, a length 72.0 m, and a width of 13.5 m. These dimensions were recently changed by GANT into a draught of 1.50 m, a length of 72.0 m and a width of 15.0 m. For a chamber width of 17.0 m, the lateral clearance between the ship and the lock chamber walls remains with 1.0 m. The required clearance above highest navigable water level is 13 m. Cargo convoys may have a maximum length of 110 m. The usable length of the navigation lock chamber shall accommodate two design ships or a cargo convoy and another smaller ship for one locking cycle. The navigation lock chamber length was determined similar to the Esna navigation lock by providing a 160 m distance between arrester positions.

The bottom of the navigation lock chamber was set at elevation 54.90 m asl, i.e. 3.0 m below the tailwater level for a minimum river discharge of 350 m³/s. The 3.0 m allow for a draught of 1.80 m, a tailwater level reduction over the following 50 years by 0.70 m and a remaining minimum keel clearance of 0.50 m. These dimensions were confirmed by the MWRI.

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### 5.4.6 Upstream and Downstream Approaches to the Navigation Locks

In order to maintain navigation during construction of the closure embankment of the diversion canal, the bottom level of the upstream approach to the navigation locks has been set at 56.60 m asl. This allows for the period of construction with a river discharge of 710 m³/s at a tailwater of 58.90 m asl to pass ships with 1.80 m draught at 0.50 m keel clearance. Only for some weeks during the closure period of the irrigation systems the river discharge would be reduced to 350 m³/s.

The preparation zones with berthing facilities have a length of 220 m. This length complies with the standards for upstream and downstream approaches to navigation locks in the main European waterways.

With an approximate width of 90 m between the mooring posts on each side of the downstream preparation sections, there is sufficient space for ships to berth on the landside whilst ships pass in two single lanes in upstream and downstream direction. The width of the upstream preparation section is uncritical, as there are no mooring posts provided on the side of the sluiceway approach. The headpond constitutes a much wider approach area then the downstream river section.

## 5.5 Freeboard Requirements

### 5.5.1 For Structures

Freeboard above Normal Operation Level at structures, such as the barrage and the dikes shall allow for headpond surcharge during design and emergency flood, wind set-up of the water surface, wave run-up on sloping embankments or on vertical walls, surge as a result of load rejection and include an additional safety margin. The safety margin for the design flood (10,000 years return period) is 1.50 m whereas for the emergency flood a safety margin of 1.0 m is applied. The surcharge was finally determined in the hydraulic model tests. The freeboard components for the New Naga Hammadi Barrage are summarised in **Table 5.2.**

**Table 5.2: Freeboard Components for the New Naga Hammadi Barrage**

| Case | NOL | Surcharge | Wind Set-up | Wave Run-up | Transient Upsurge | Safety Margin | Required Crest Level | Available Safety Margin |
|---|---|---|---|---|---|---|---|---|
| | m asl | m | m | m | m | m | m asl | m |
| Normal Operation | 65.90 | 0.00 | 0.11 | 0.72 | 0.30 | 1.50 | 68.53 | 1.97 |
| Emergency Release (*) | 65.90 | 1.15 | 0.10 | 0.72 | 0.00 | 1.00 | 68.87 | 1.13 |

(*): Only sluiceway operating

Freeboard components due to the action of wind were calculated with the maximum average one hour wind speed of 65 km/h measured over land at the Sohag meteorological station.

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The Crest level of the barrage structures was finally adopted with 69.0 m asl.

## 5.5.2 For Sluiceway

The freeboard requirement is related to the calculation of extreme wind set-up (0.11 m) and transient upsurge (0.30 m), from which the freeboard for the sluiceway gates has been set to 0.40 m. With the normal headpond level at 65.90 m asl, the gate crest level results in 66.30 m asl. This is shown on **Album Drawing no. 127** for the sluiceway gate including the flap.

At the upstream side, 5 stoplog elements (2.70 m each, in total 13.50 m, sill elevation 52.80 m asl) reach up to elevation 66.30 m asl and provide 0.40 m freeboard.

Maintenance works shall be carried out up to a discharge of 2,400 m³/s equivalent to a tailwater level of 61.80 m asl. At the downstream side, 6 stoplog elements (2.70 m each, in total 16.20 m, sill elevation 46.00 m asl) reach up to 62.20 m asl and provide 0.40 m freeboard.

## 5.5.3 For Navigation Lock

For the navigation lock, the emergency flood water level (7,000 m³/s = 76.05 m asl) determines the maximum height of the upstream head. With no freeboard needed for this extreme case, the maximum elevation of the upstream gate shall be 67.10 m asl. The associated downstream water level is 66.50 m asl, requiring a gate crest elevation of 66.60 m asl. The required heights of the mitre gates are 5.10 m for the upstream and 11.60 m for the downstream.

For the maintenance case, the relevant upstream headpond level is 65.90 m asl, while the relevant downstream tailwater level for maintenance is 61.80 m asl.

The stoplogs arrangement provide the following crest levels:

- Upstream head:

- upstream stoplogs (sector gate):     $56.60 + 4 * 2.70 = 67.40$ m asl.

- upstream stoplogs (mitre gate):     $62.00 + 2 * 2.70 = 67.40$ m asl.

- downstream stoplogs:     $54.90 + 3 * 2.70 = 63.00$ m asl.

The respective freeboards are 0.50 m upstream and 1.20 m downstream.

- Downstream head:

- upstream stoplogs:     $52.90 + 5 * 2.70 = 66.40$ m asl.

- downstream stoplogs:     $54.90 + 3 * 2.70 = 63.00$ m asl.

The respective freeboards are 0.50 m upstream and 1.20 m downstream.

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## 5.6    Surges by Powerplant Operation

When the steady flow in the Nile River is altered by sudden shut-off of turbine units at the powerplant, a surge is created which probates in both upstream and downstream directions. Upstream of the powerhouse the water level rises instantaneously when the discharge is reduced. Downstream a negative surge occurs which drops the water level. With distance from the powerplant, surges become flatter due to spreading and energy dissipation.

The case under which extreme surges are generated is closure of the units after complete load rejection. To quantify the transient phenomena with the surge control mode, numerical simulations were carried out using the computer program CARIMA, which simulates unsteady flow in open channels.

Upstream of the New Barrage, transient flow in an 80 km long reach of the Nile river was simulated assuming the powerhouse to operate in the sailing mode subsequent to load rejection at maximum powerhouse discharge. For the assumed maximum powerhouse discharge of 1,670 m³/s at NOL of 65.90 m asl the discharge in the event of load rejection was assumed to reduce to 835 m³/s (sailing mode) in 4.5 seconds. Within 2 minutes the sluiceway re-establishes the initial total discharge. The height of the surge at the powerhouse and close to the adjacent structures will reach some 0.30 m above NOL. The surge is attenuated to a height of less than 0.10 m within a distance of 5 km upstream.

Downstream of the new site, transient flow in a 35 km long reach of the Nile river was simulated with the above conditions. The calculated maximum negative surge, close to the structure, was some 0.29 m. Attenuation of the wave takes place over a distance of about 4 km downstream, at which point the residual wave height will not exceed 0.10 m. These calculated extreme surge values have been considered in the design.

## 5.7    Head Regulators for Main Irrigation Canals

The structural rehabilitation measures at the Eastern and Western Irrigation Canals include to replace the upper gates leaves with fixed concrete slabs. Therefore, the outflow will be controlled by vertical gates with submerged flow conditions. The discharge will be through a number of gate openings of identical size.

The maximum discharge defined through irrigation demands has to pass the structure at n-1 conditions, i.e. mal function of one gate, at minimum observed/expected head difference.

The downstream water level needs to be at least 0.25 times the gate opening above the gate in order to provide sufficient submergence and to avoid development of irregular oscillations of discharge and downstream submergence, which could induce vibrations and thus harm the structures at a certain frequency.

Details on discharge capacities and numbers of required openings are given in **Volume 2, Section 13, Appendix 13.1** "Summary of Daily Discharge Data and Head Regulators Discharge Capacity".

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## 5.8    Riprap Protection Works

### 5.8.1    General

Riverbed and embankment protection is limited to an area upstream and downstream of the new Barrage, to prevent scouring and regressive erosion towards the individual structures. Undue deposition of bed material has also to be avoided in order to maintain safe navigation and continuous powerhouse operation.

Experience has shown that dumped riprap is the most suitable and economic type of slope and bed protection [6]. Therefore, the proposed riverbed protection works are based on application of riprap, together with appropriate filter layers and geotextiles. The upper riprap layer is subject to attack by the stream flow phenomena, and its stability has been investigated in the hydraulic model tests by the Barrage Model and in the Sluiceway Detail Model. The dimensioning of the lower transition layers is subject to application of filter criteria from approximate literature.

### 5.8.2    Principles of Hydraulic Dimensioning

#### (1)    Background

The following impacts are relevant for the design of the upper layer of a riverbed protection:

- flow related impacts:              flow velocity and turbulence,
- navigation related impacts:        waves, propeller jet and re-circulation,
- operation and wind related impacts:    surges and wind waves.

The final approach in the tender design for dimensioning the upper layer of riverbed protections was composed of the following steps:

1) Hydraulic model tests to verify the sizes and extension of the individual protection types, as pre-dimensioned by empirical approaches.

2) Further empirical approaches were used to compensate the results from 1) for hydraulic model scale effects. Thereby, the impact of waves on slopes and navigation on the riprap was considered.

3) The extension of riprap types for the upper layer of river protection was re-defined with all necessary correction.

Riverbed protection is different for permanent structures and for temporary structures such as the diversion canal, or the up- and downstream cofferdams.

In the riprap design, the failure mechanism under loads acting on the riverbed and embankments, during construction and operation, were considered. Typical loads and responses are flow currents, turbulence, wave action, and displacements relative to the as-built position.

Further relevant for the design of the riverbed protection in the approaches, harbours, berthing areas, are the impacts from navigation, i.e. from a propeller jet. In the left navigation lock approaches, when the lock is opened for flood evacuation, this constitutes the critical condition.

For embankment protections it is distinguished between areas permanently exposed to water, and higher levels which only are exposed to wave run-up or to temporary water level rise during flood releases.

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## (2) Properties of Riprap

One of the possible sources of riprap which might be used for riverbed protection is the Esaweia quarry. The results of specific investigations on properties of material considered suitable for riprap and rockfill at the New Naga Hammadi Barrage are summarised in **Volume 4, Section 4.23** and **Annex 1**.

### (2.1) Grading of Riprap

Information on the gradation and mass distribution of rock and granular material is needed to assess the stability of armouring stone, filter rules, selection of a construction method and the equipment involved. The particle mass distribution is presented by cumulative curve where $W_{50}$ expresses the particle mass above which 50% of the sample is of lighter particles. $W_{85}$ and $W_{15}$ are defined correspondingly. The overall steepness of the curve indicates the grading width, normally given by the $W_{85}/W_{15}$ or by the $D_{85}/D_{15}$ ratio. $D_{15}$ and $D_{85}$ are important indicators for the design of filters. Ranges recommended for the gradation widths are summarised in **Table 5.3**.

**Table 5.3: Ranges of Gradation Widths**

| Gradation | $D_{85}/D_{15} = (W_{85}/W_{15})^{1/3}$ | $W_{85}/W_{15}$ |
|---|---|---|
| Narrow | 1.2 – 1.5 | 1.7 – 3.4 |
| Wide | 1.5 – 2.5 | 3.4 – 16.0 |
| Very Wide | 2.5 – 5.0 | 16.0 – 125 |

### (2.2) Riprap Classification by Size

The term riprap usually applies to armouring stone with a wide gradation which are generally bulk placed and used in revetments. In several countries including Egypt, standard grading classes are defined. The proposed standard gradation for armouring stones is relatively narrow. For the New Naga Hammadi, six grading classes (**Table 5.4**) were defined in line with international practice, which then were investigated in the hydraulic model for their stability under the attached flow.

**Table 5.4: Riprap Classification used for New Naga Hammadi**

| Riprap Type | Mean Diameter $D_{50}$ m | Mean Weight $W_{50}^{(*)}$ kg |
|---|---|---|
| R1 | 0.74 | 620 |
| R2 | 0.57 | 315 |
| R3 | 0.39 | 100 |
| R4 | 0.26 | 30 |
| R5 | 0.17 | 8.5 |
| R6 | 0.10 | - |

Note: (*) Details on conversion between diameter and weight of stones are given in **Paragraph 2.3** below.

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## (2.3)   Specification of Riprap by Weight of Particles

Resistance to erosion of riprap particles is governed by their weight, their shape and the interlocking effect with other particles. Riprap is specified in terms of stone weight rather than particle diameter in order to eliminate the variable effect of densities and particle shapes of the materials obtained from quarries. Based on the results of the hydraulic model tests, 6 riprap types were determined for the surface layers of the six riverbed protection types. The model riprap which proved to be stable under the flow conditions simulated in the hydraulic model was analysed with regard to its grain size distribution (gradation parameter $D_{15}$, $D_{50}$, and $D_{85}$), and the dry density of the model riprap.

The riprap used in the hydraulic model tests has an angular shape. Samples of riprap type R1 were analysed by measuring the length in 3 axes and the weight of particles. The following deals with the assumptions made and the approach applied in transforming the properties from the model riprap to prototype conditions.

The measures on particles which easily and objectively can be made are:

- $z$ = sieve size (smallest square hole which a particle can pass)

- $L$ = maximal axial length

- $d$ = thickness (minimum axial length)

The following average ratios deducted from measurements over a range of applicable rock types and sizes after transport and handling are according to [5]:

- $L / d = 2.0$,

- $d / z = 0.75$, and

- $L / z = 1.50$.

In the approach to the riprap design, the sieve size $z$ is set equal to $D_{50}$. It is further assumed that the weight of the riprap particles can be sufficiently described by the sieve size $z$ (or $D_{50}$) and a shape factor SF. The shape factor was derived from geometry after statistical analysis of the model riprap. In accordance with the above average ratios of the dimensions of riprap, and a dry density of the model riprap of 2,200 kg/m³ the weight of the riprap particle is:

$$W_{50} = SF \cdot 1.0 \, D_{50} \cdot 0.75 \, D_{50} \cdot 1.5 \, D_{50} \cdot 2{,}200 \text{ kg/m}^3 = SF \cdot 1.125 \, D_{50}^{3} \cdot 2{,}200 \text{ kg/m}^3$$

The shape factor of the model riprap was derived from a statistical analyses of 75 particles taken from the model riprap type R1:

$$SF = 0.778.$$

This is valid for an 80% confidence interval (i.e. 80% of the analysed particles had a shape factor below 0.778 and only 20% stones above).

Accordingly the stone weight can be approximated from $D_{50}$ by means of the following formula:

$$W_{50} = 0.78 \cdot D_{50}^{3} \cdot 2{,}200 \text{ kg/m}^3$$

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## (2.4) Gradation of Riprap by Weight

The gradation by weight is based on the gradation given by size for uniformity coefficients. The ratios applied are summarised in **Table 5.5**.

### Table 5.5: Principles for Riprap Gradation by Weight

| Riprap Type | Uniformity Coefficient U | Size Ratio $D_{85}/D_{15}$ | Weight Ratio $W_{85}/W_{15}$ |
|-------------|--------------------------|----------------------------|------------------------------|
| R1 | 1.47 | 2.15 | 10 |
| R2 | 1.47 | 2.15 | 10 |
| R3 | 1.70 | 2.9 | 25 |
| R4 | 1.70 | 2.9 | 25 |
| R5 | 1.70 | 2.9 | 25 |
| R6 | 5.00 | 25 | - |

The riprap types and their gradation specified by weight are given in **Table 5.6**.

### Table 5.6: Specification of Riprap Types by Weight

| Riprap Type | $W_{15}$ kg | $W_{50}$ kg | $W_{85}$ kg |
|-------------|-------------|-------------|-------------|
| R1 | 245 | 620 | 1,960 |
| R2 | 110 | 315 | 900 |
| R3 | 22 | 100 | 475 |
| R4 | 6.5 | 30 | 140 |
| R5 | 1.8 | 8.5 | 40 |

### 5.8.3 Empirical Design Approaches

### (1) Stability Criterion based on Phenomena measured in the Hydraulic Model

From velocity measurements in the hydraulic model, it was seen that close to the structures the vertical and horizontal velocity distributions were distorted due to 3-dimensional flow phenomena. Specifically, these locations required a good protection. Hence, an empirical approach was applied which takes into account the phenomena detected by the hydraulic model tests.

A stability criterion described in [2] was derived from equilibrium conditions of the attacking momentum of the flow against the particle, and by its own weight. The stability coefficient $B'_{lim}$ was determined to differentiate between a stable and an unstable range:

$$B'_{lim} = 1.25...1.30 = \frac{v_{Bottom}(1 + 3\,SDV)}{\sqrt{d_{50} \cdot g \cdot \Delta'}}$$

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where:

$v_{Bottom}$      = $v_{z\,=\,0.1\,d}$, velocity measured at 10% of the water depth in m/s

$\Delta'$      = submerged relative density of the riprap

SDV      = standard deviation of the velocity fluctuations

$d_{50}$      = mean diameter of the riprap type applied in m

The hydraulic model tests in the Barrage and the Sluiceway Detail Model were then used to measure the parameters necessary for calculation of the actual stability coefficient versus distance from the New Barrage structures and distribute the different riprap classes.

## (2) Attack on Embankments by Wind Generated Waves

Wind speed, fetch length and the shape and depth of the free water surface are the major factors influencing the height of the waves. Upstream and downstream of the New Barrage there prevail different conditions for generation of wind waves, in particular with regard to the fetch length and depth of water.

### Table 5.7: Wind Wave Generating Conditions at the Barrage

| Location | Fetch Length m | Depth of Water m | Design Wind Speed km/h | Significant Wave Height m |
|---|---|---|---|---|
| Headpond | 1,700 | 11.0 | 65 | 0.32 |
| Tailwater | 5,500 | 9.5 | 65 | 0.50 |

Significant wave heights were calculated applying the spectral method. For an assumed riprap density of 2,200 kg/m³ (e.g. from Esaweia Stone) placed on 1:3 bank slopes, the required riprap size was calculated by the HUDSON formula. Results for a stability factor of 2.5 are given in **Table 5.8**.

### Table 5.8: Riprap Sizes for Embankment Slopes 1:3 depending on Significant Wave Height

| Significant Wave Height m | $D_{min}$ m | $D_{50}$ m | $D_{max}$ m | $W_{50}$ N | $M_{50}$ kg |
|---|---|---|---|---|---|
| 0.30 | 0.07 | 0.14 | 0.22 | 45 | 4.6 |
| 0.35 | 0.08 | 0.16 | 0.26 | 71 | 7.3 |
| 0.40 | 0.09 | 0.19 | 0.29 | 107 | 11 |
| 0.45 | 0.10 | 0.21 | 0.33 | 152 | 15 |
| 0.50 | 0.12 | 0.23 | 0.37 | 208 | 21 |
| 0.55 | 0.13 | 0.25 | 0.40 | 277 | 28 |
| 0.60 | 0.14 | 028 | 0.44 | 360 | 37 |

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The riprap characteristics selected by the wave height criterion are given in **Table 5.9**.

### Table 5.9: Riprap Size for Slopes

| Location | Significant Wave Height m | Required $D_{50}$ m | Selected Riprap Type class, m |
|---|---|---|---|
| Upstream | 0.32 | 0.15 | R5, D50 = 0.17 > 0.15 |
| Downstream | 0.50 | 0.23 | R4, D50 = 0.26 > 0.23 |

### (3)   Riverbed Protection against Navigation Impacts

Navigation has two types of impacts on the stability of riverbed and slopes, by:

- Navigation induced waves, which may cause local instability of the embankment slopes, and
- Propeller jets, which may cause local instability of the riverbed protection.

### (3.1)   Navigation Induced Waves

The height of navigation induced waves depends mainly on the speed of the ships, the width of the navigation channel and the proximity of the ships to the river banks.

The speed of the ships in the vicinity of the New Naga Hammadi Barrage, i.e. in the preparation zones or berthing areas, will not exceed some 6 km/h, and the induced waves will not exceed a height of 0.5 m. Hence, the riprap type R4 is considered to be sufficiently stable with regard to navigation induced waves.

### (3.2)   Propeller Jet Attack to the Riverbed

It is conservative to assume that for the navigation fleet on the Nile maximum propeller exit velocities are about 6.0 m/s and propeller diameter are 0.8 m, corresponding to European inland crafts.

Over a distance of about 2.6 times the diameter from the propeller, the velocity of the propeller jet remains constant. Further away, the jet velocity reduces both in jet direction and with distance from the jet axis, with an opening angle of the propeller jet of about 12-13°. The velocity of the propeller jet which may attack the riverbed can be estimated by [3]:

$$v_{max,bed} = v_0 \cdot E \cdot \left(\frac{h_p}{D}\right)^{-1.0}$$

where
$v_{max,bed}$ = maximum jet velocity at the riverbed, m/s

$h_p$ = vertical distance between propeller axis and riverbed , m

$D$ = Propeller-diameter, m

$E$ = coefficient, inland navigation craft, E = 0.42

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## (3.3)  Conditions in the Headpond

The characteristic depth of water in the navigation fareway of the new headpond between water surface level of 65.90 m asl and riverbed level of 56.60 m asl will be 9.3 m. Under such conditions, the propeller jet will reach the riverbed at a distance of not less than 34 m, and the residual velocity of the propeller jet will be as low as 5% of its maximum, i.e. not higher than 0.3 m/s. Therefore, the impact of propeller jets on the riverbed in the navigation fareway of the headpond is negligible.

The governing criterion for selection of adequate riprap size in the berthing areas is the required stability during flood evacuation through the left navigation lock chamber. It is foreseen to place riprap which successively increases enforcing towards the navigation lock, starting with riprap type R5 some 270 m upstream of the navigation lock, and ending with type R2 at the navigation lock upper head.

## (3.4)  Conditions in the Diversion Canal

During construction, river navigation will continue through the diversion canal which has a bed level of 52.0 m asl. The available depth of water of 7 meter will prevent high currents caused by moving ships to reach the canal bed. The residual velocity of the propeller jet reaching the riverbed will not exceed 0.5 m/s. Therefore, navigation induced phenomena are not relevant for the selection of the riprap size. Damage of protections in the headpond from construction (e.g. from placement of closure dam material) could be repaired prior to impounding.

## (3.5)  Conditions in the Tailwater

In the downstream approach to the navigation lock, stopping and starting of ships will be as frequent as in the headpond, however, water depth is less. At the given minimum river discharge of 350 m³/s (tailwater level of 57.90 m asl) the depth of flow will be 3.0 m. The jet velocity reaching the riverbed has a residual value of 20 - 25% of the maximum jet velocity, i.e. about 1.2 to 1.5 m/s. Critical conditions may arise in the area of the navigation lock guide wall only for conditions when a propeller jet touches the guide wall and is diverted directly towards the riverbed protection. Therefore, the guide wall needs a toe protection to prevent scouring.

The riprap size in the downstream approach to the navigation lock and the preparation area depends on the emergency case of flood evacuation through the left navigation lock chamber. Riprap type R3 will be required in the berthing areas, and type R2 in the vicinity of the navigation lock lower head.

### 5.8.4  Hydraulic Model Tests

The stability of the permanent riverbed protection by riprap was investigated in hydraulic model tests for river discharges between 1,840 and 7,000 m³/s and relevant project operation modes. The temporary diversion canal was tested for discharges between 800 and 2,900 m³/s. The extent of the different sizes of riprap was verified with the tender design, and resulted in the final arrangement of riprap protection as given in **Album No. 22**.

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During the model tests, two methods were used to check the stability of the riprap:

- Observation of riprap particles during the tests, and subsequent inspection of the riprap areas after completion of each test. Riprap classes were marked by different colours. The riprap was considered to be stable if, counting the displaced stones, transport of particles was less than 1% of the riprap area considered, and scouring did not occur;

- Measuring of the flow velocity at the bottom (0.1 of water depth), or calculation of the bottom velocity from measured velocity distributions, and their application to empirical stability criteria.

Incipient motion of particles is commonly described by the Shields-diagram (**Figure 5.4**). As long as the Reynolds number of the riprap grains in model and prototype are less than critical, the observations from the hydraulic model are applicable without being subject to scale effects. This is the case for grain Reynolds numbers above 150, i.e. for particles larger than about 8 mm in the model (representing 0.24 m in prototype), i.e. riprap types R1 to R4. For smaller riprap (types R5, R6) stability is to be confirmed by appropriate empirical approaches.
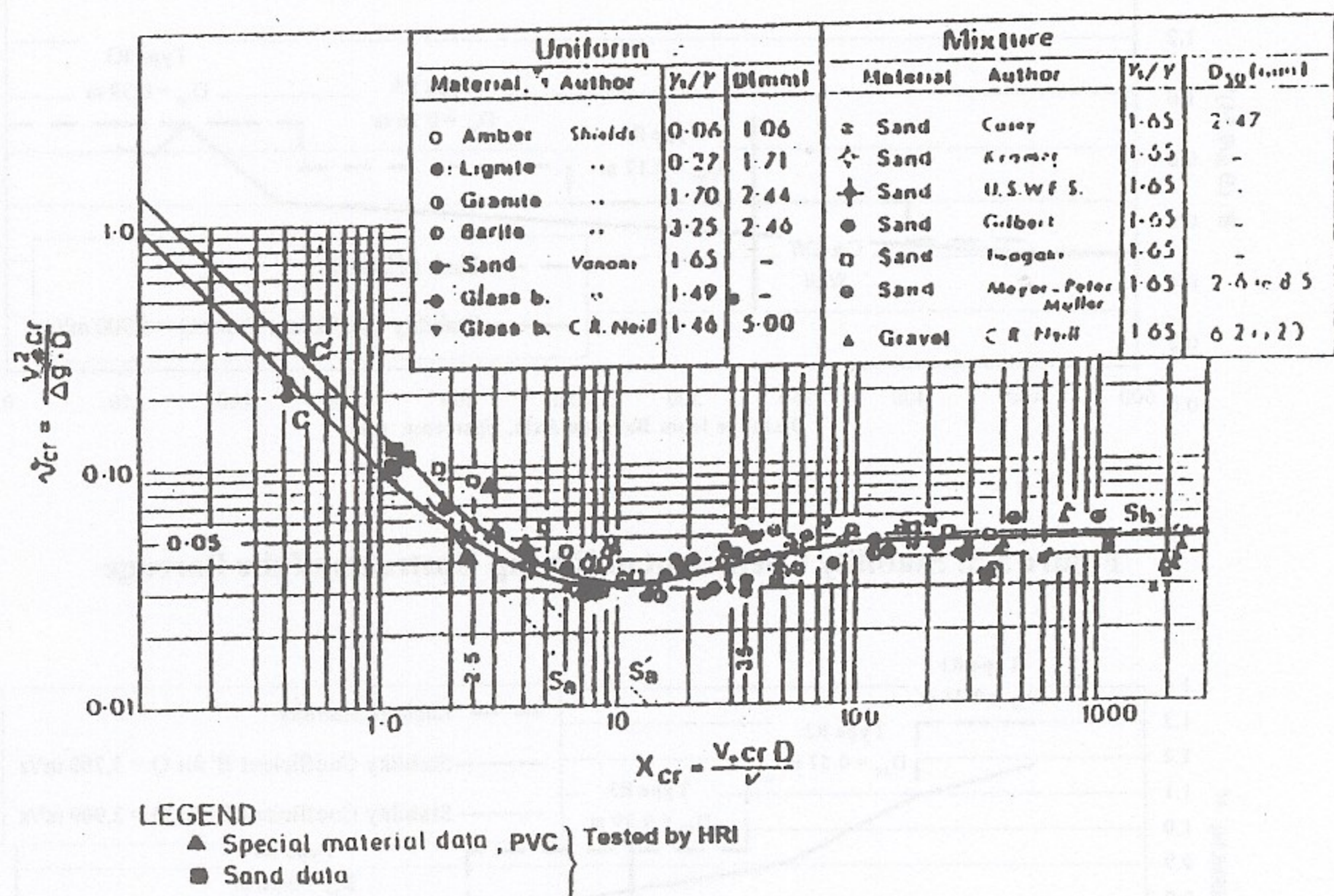


Figure 5.4: SHIELDS Diagram for Incipient Motion of Particles

The particular areas where riverbed protection by riprap has been thoroughly investigated in the model, are upstream and downstream of the sluiceway, in the exit from the powerhouse, and in the diversion canal. For details on the hydraulic model tests it is referred to the separate reports on hydraulic model tests [2] and [3].

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### 5.8.5 Extent and Type of Riverbed Protection by Riprap

It is a requirement for guaranteeing the long-term stability of the permanent concrete works, that the riprap protection within the area surrounded by the diaphragm wall of the construction pit plus an additional coverage of about 150 meter downstream from the wall should be absolutely stable for all admissible discharge operation modes, up to the evacuation of the emergency flood (7,000 m³/s). The adjacent river bottom reaches upstream and downstream shall be safely protected for release of the 100 years flood discharge, i.e.2,900 m³/s, which has never happened since the implementation of the HAD.

The riprap riverbed protection is placed over the full width of the river channel. At the transition from the riverbed protection to the original riverbed, scouring is most likely to occur. These transition zones are designed to prevent propagation of scour towards the diaphragm wall remaining from the construction pit, see **Section 7.3.3** of this Tender Design Report.

Based on the model tests and velocity measurements, the actual and required stability coefficients for the upstream and for the downstream reaches are presented in **Figures 5.5** and **5.6**.
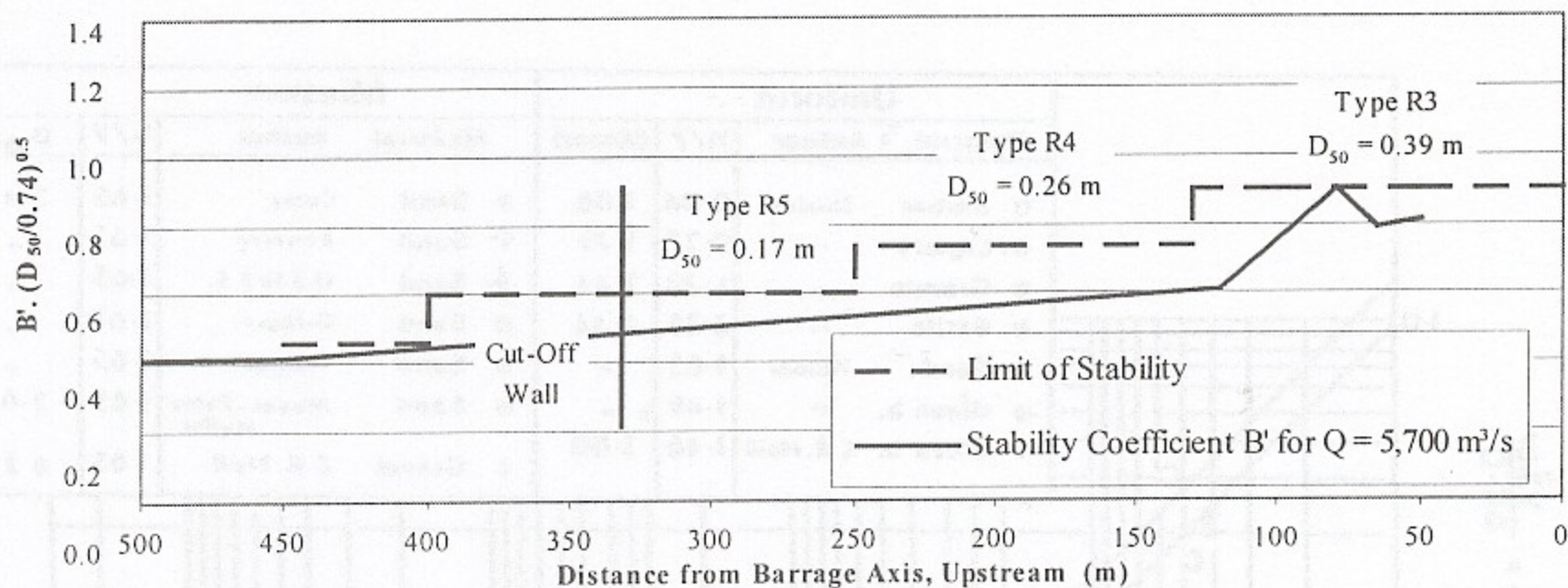


**Figure 5.5: Stability Coefficient of Riprap Upstream of the Barrage**
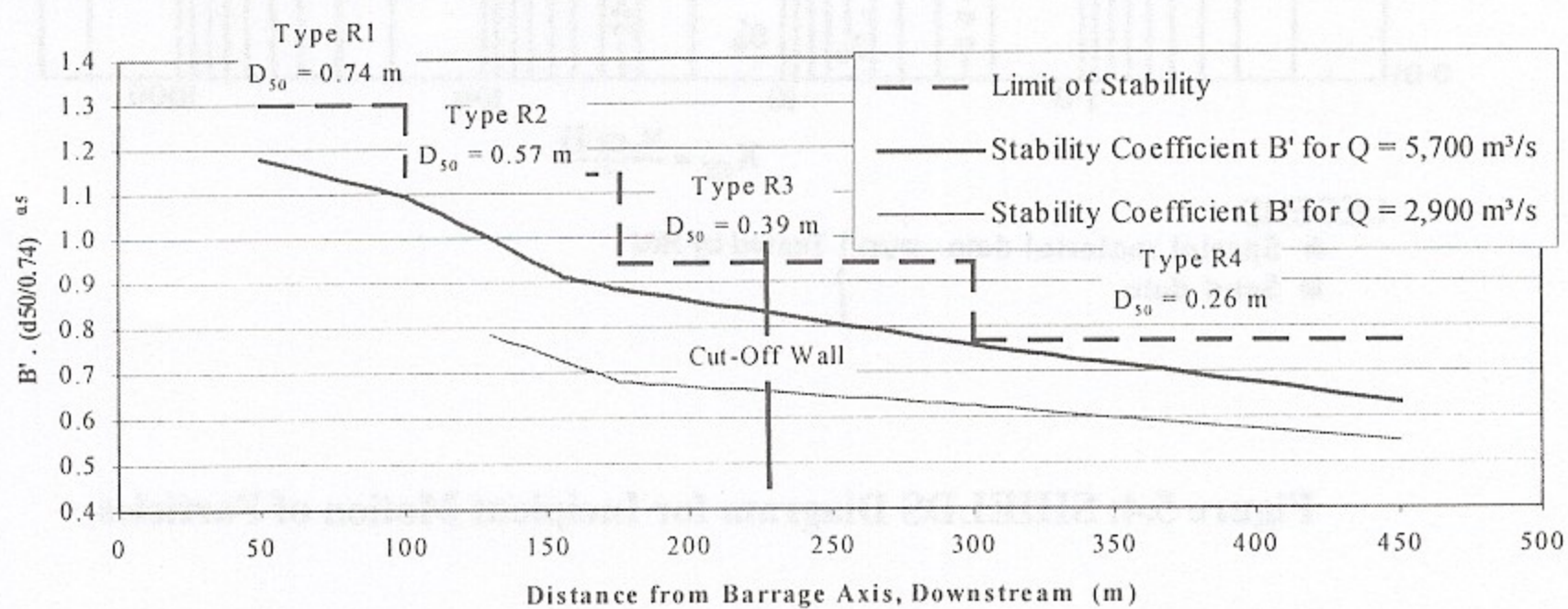


**Figure 5.6: Stability Coefficient of Riprap Downstream of the Barrage**

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The six for New Naga Hammadi defined riprap types R1 to R6 shall be placed for riverbed and embankment protection in the areas briefly described in the following and shown on **Album No. 24.**

### (1) Riprap Type R1: $D_{50} = 0.74$ m or $W_{50} = 620$ kg

The heaviest riprap type is placed at the area which is subject to frequent and continuos high flow velocities, i.e. immediately downstream of the powerhouse exit bay adjacent to the draft tubes. The maximum flow velocity of some 2.6 m/s will prevail over the entire depth. In addition, intense bulk turbulence and spiral type flow are expected. The riprap type R1 in the exit bay will cover an area of some 50 m in flow direction and between 75 and 90 m perpendicular to it.

In case that the riverbed protection is arranged by placement of single stones of rather uniform size then riprap stone size type R2 can be applied instead, see **Section 7.3.5** of this Tender Design Report.

### (2) Riprap Type R2: $D_{50} = 0.57$ m or $W_{50} = 315$ kg

Riprap type R2 will be applied to the downstream area over some 125 m in flow direction. This riprap type will cover a width of 270 m with the areas downstream of sluiceway and powerhouse. Additionally the navigation locks approaches, extending some 55 m from the lower head shall have the same type of riprap. In the headpond area only the approach of the left navigation lock shall be protected by type R2 riprap, extending some 52 m from the upper head. Riprap type R2 is also applied as a 6 m wide toe protection along the intermediate pier between powerhouse and sluiceway and in front of the right abutment pier of the sluiceway.

### (3) Riprap Type R3: $D_{50} = 0.39$ m or $W_{50} = 100$ kg

Riprap type R3 shall be placed in the areas downstream and adjacent to type R2. It covers an area of some 250 m on the left bank downstream of the powerhouse, and an area of some 125 m length downstream of powerhouse and sluiceway. It covers also the berthing areas over a width of some 100 m and a length of 400 m. The slopes on the right bank within the berthing area are protected by type R3.

In the upstream area, riprap type R3 shall be placed over the entire 310 m width of sluiceway and powerhouse, and extend some 130 m upstream. This includes also the corresponding part of the left bank slope. A length of some 80 m upstream of the type R2 protection in the navigation lock approach and the entrance of the right navigation lock shall also be protected by type R3.

### (4) Riprap Type R4: $D_{50} = 0.26$ m or $W_{50} = 30$ kg

Riprap type R4 shall be placed downstream and adjacent to the type R3 for some 150 m over the entire width. The remaining right bank slope of some 230 m length shall be covered by type R4 riprap.

The downstream face of the permanent closure dam shall also be of the type R4, designed against the attack by wind induced waves.

An area of approximately 1,200 m$^2$ between the downstream end of the right abutment pier of the sluiceway and the navigation lock shall be protected by the same type R4.

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In the headpond, for some 120 m in flow direction upstream of the type R3 protected area, the entire river width of some 350 m between left bank and navigation lock guide wall shall be protected by type R4. The upstream navigation lock approach area is also covered by this riprap type for another 60 m upstream of the type R3 protection. Slopes on the right bank are protected by riprap type R4.

## (5)  Riprap Type R5: $D_{50} = 0.17$ m or $W_{50} = 8.5$ kg

Riprap type R5 is placed for some 150 m length across the entire river channel for riverbed protection when approaching from upstream including a length of 60 m in the navigation lock approaches before the transition to R3. Both left and right bank slopes of the temporary diversion canal shall also be protected by type R5 riprap, designed to resist against wave impact for a significant wave height which is reduced due to diffraction effects. The slope of the permanent part of the upstream left bank shall be protected by type R5.

## (6)  Riprap Type R6: $D_{50} = 0.10$ m

Riprap type R6 shall be used for the temporary works of the diversion canal, i.e. riverbed total length and left bank slope upstream of the closure dam.

Bottom flow velocities deducted from hydraulic model tests resulted in approximately 1.3 m/s. With the turbulence factor SDV of 0.12, a Stability Coefficient of $B' = 0.60$ was derived. This is below the limit of $B'_{lim} = 0.68$, which takes into account primary erosion and formation of an armouring layer on the bed of the fresh excavated.

Considering that the highest flow velocities were measured in a limited area and outlet close to the left bank in the return section of the canal, the riprap protection can be considered to be safe at both the canal bed and the banks. In case that during operation of the diversion canal minor erosion is observed, the affected area could later be reinforced by riprap type R5.

The bank slope within the downstream berthing area facing the flood channel shall be of type R6 as well as the top of the platform on elevation 65.0 m asl downstream and to the left of the powerhouse.

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## 5.9 References

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# CHAPTER 6

# APPROACH TO DESIGN AND STABILITY OF THE MAIN CONCRETE WORKS

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# 6. APPROACH TO DESIGN AND STABILITY OF THE MAIN CONCRETE WORKS

## 6.1 General

The main concrete works which have been considered in stability calculations and samples of structural design are:

- the navigation locks
- the sluiceway
- the powerhouse
- the powerhouse abutment pier
- the public traffic road bridge

Other components of the main concrete works were developed to the same stability principles, and the stability against uplift has been checked by the quantities resulting from the dimensions applied in design. A detailed stability analysis will be performed during construction design, which then will consider all sections of the structures between expansion joints, plus the interference from shear keys between structural blocks.

Sample structural calculations were made to estimate the reinforcement contents of a typical section of the above cited structures.

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## 6.2     General Design Load Data and Calculation Methods

### 6.2.1    Dead Loads

Dead loads are calculated based on the unit weights given in **Table 6.1**.

<p align="center"><strong>Table 6.1: Unit Weight for Dead Loads</strong></p>

| Material | Unit Weight kN/m³ |
|---|---|
| Steel, cast steel | 78.5 |
| Cast iron | 72.5 |
| Aluminium alloys | 28 |
| Timber (treated or untreated) | 8 |
| Reinforced concrete | 25 |
| Concrete without reinforcement | 23.5 |
| Cement mortar | 21.5 |
| Water ($\gamma_w$) | 10 |
| Asphalt pavement | 23 |
| Bituminous material | 11 |
| Loose sand, earth, and gravel <br> • dry unit weight $\gamma_b$, <br> • saturated unit weight[1] $\gamma_r$ | <br> 17 <br> 19 |
| Compacted sand, earth and gravel <br> • dry unit weight $\gamma_b$ <br> • saturated unit weight[1] $\gamma_r$ | <br> 19 <br> 21 |

The weight of permanent parts of non-structural attachments (like platforms) or of mechanical equipment which cannot be removed (like crane rails, guide rails, piping or embedded parts) is considered as dead load.

The thickness of a tiled (or similar) floor finishing is generally assumed to be 7 cm and the corresponding dead load is being considered as follows[2]:

$$0.07 \text{ m} * 21.5 \text{ kN/m}^3 = 1.5 \text{ kN/m}^2$$

---

[1]   submerged unit weight:   $\gamma' = \gamma_r - \gamma_w$

[2]   to be considered for the powerhouse and auxiliary building only

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## 6.2.2   Earth Pressure Loads

### (1)  Active Earth Pressure

The assumption that *active earth pressure* is acting on a retention structure is justified, if one of the following conditions is provided in the considered case:

- rotation around footing point:   displacement at the top > 0.002 · height of the structure;

- rotation around the top:   displacement at the footing point > 0.005 · height of the structure;

- parallel displacement:   displacement > 0.001 · height of the structure.

Earth pressure loads acting on the *active side* of retention structures which allow for the following are being calculated as follows:

$$P_{ea} = 0.5 \left( \gamma' \, H_{ea} + \frac{2 \, Q \sin \alpha_a}{\sin (\alpha_a + \beta_a)} \right) H_{ea} \cdot k_a \tag{6.1}$$

Where:

$P_{ea}$ = total active earth pressure in kN/m acting on a retention structure (per meter run in longitudinal direction); the inclination measured clockwise from perpendicular to the wall can be assumed as $\delta_a = 2/3 \, \varphi'$ (whereby $\varphi'$ means friction angle of the backfill), with an downward orientation towards the active side of the wall;

$H_{ea}$ = height of backfill behind a retention structure;

$\gamma'$ = submerged unit weight: $\gamma_r - \gamma_w$;

$\gamma_r$ = saturated unit weight of soil, kN/m$^3$;

$\gamma_w$ = unit weight of water, kN/m$^3$;

$\alpha_a$ = inclination of a retention wall on which active earth pressure is acting (measured clockwise from horizontal);

$\beta_a$ = inclination of the ground surface behind a retention wall (measured clockwise from horizontal);

$k_a$ = coloumb coefficient for active earth pressure, whereby $k_a \geq 0.15$.

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## (2) Pressure at "Earth at Rest"

For structures subject to earth pressures which are not allowing displacements (because they are not free to slide or to deflect), the case "earth at rest" has to be considered, whereby the corresponding k-value is named $k_0$ and calculated as follows:

$$k_0 = 1 - \sin \varphi'$$ 

(6.2)

## (3) Compaction Effect

On the safe side, the compaction effect is covered by case "earth at rest".

## (4) Passive Earth Pressure

Earth pressure loads acting on the *passive side* of retention structures are being calculated as follows:

$$P_{ep} = 0.5 \, \gamma' \, H^2_{ep} \, k_p$$ 

(6.3)

Where:

$P_{ep,s} =$ total passive earth pressure in kN/m acting on the passive side of a retention structure (per meter run in longitudinal direction); the inclination measured clockwise from perpendicular to the wall can be assumed as $\delta_p = 1/3 \; \varphi'$ (whereby $\varphi'$ means friction angle of the backfill), with an upward orientation towards the passive side of the wall;

$H_{ep} =$ foundation depth of a retention structure (measured below the ground level of the passive wedge);

$\gamma' =$ submerged unit weight: $\gamma_r - \gamma_w$;

$\gamma_r =$ saturated unit weight of soil, kN/m³;

$\gamma_w =$ unit weight of water, kN/m³;

$k_p =$ coloumb coefficient for passive earth pressure.

For stability considerations, a maximum of 33% of the calculated passive earth pressure may be taken into account, if large displacements of a retention structure are not permitted. If part of the passive earth pressure is considered for the stability calculation, then it must be guaranteed that the respective soil will not be removed during the lifetime of the structure.

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### 6.2.3 Wind Loads

Wind loads have not been taken into account for the purpose of tender design of the Naga Hammadi Barrage components, but shall be applied at construction design stage (Task B).

The wind loads will then be assumed in accordance with the provisions of the "Egyptian Code for Loads and Forces on Structures and Building Works (EC 45/1996)" as follows:

- External Design Wind Pressure: $P_e = C_e \cdot K \cdot q$        (6.4)

- Internal Design Wind Pressure: $P_i = C_i \cdot K \cdot q$        (6.5)

Where:

$C_e$   = external wind pressure distribution factor [-], as given by **Table 6.3**;

$C_i$   = internal wind pressure distribution factor [-], as given by **Table 6.4**;

$P_e$   = external wind pressure, $kN/m^2$;

$P_i$   = internal wind pressure, $kN/m^2$;

$q$   = basic wind pressure $kN/m^2$, for a wind velocity of 120 km/h to be considered, the basic wind pressure is to be taken as $0.7\ kN/m^2$;

$K$   = exposure factor [-], as given by **Table 6.2** for the calculation of $P_e$.

**Table 6.2: Wind Exposure Factor**

| Height z above the ground [m] | K [–] |
|:---:|:---:|
| 0 - 10 | 1.0 |
| 10 - 20 | 1.1 |
| 20 - 30 | 1.3 |
| 30 - 50 | 1.5 |
| 50 - 80 | 1.7 |

The K factor for $P_i$ calculations is based on z equals to the half height of building.

The external and internal wind pressure distribution factors $C_e$ and $C_i$ are functions of the structure geometry and dimensions. $C_e$ values, which should be used for building with rectangular elevations are given in **Table 6.3**.

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## Table 6.3: Wind Pressure Distribution Factor $C_e$

| Structural Element | $C_e$ * |
|---|---|
| Walls on windward side | 0.8 |
| Walls on lee side | -0.5 |
| Side walls | -0.7 |
| Flat roofs | -0.8 |

\* = Negative values mean suction

$C_i$ values for buildings with rectangular elevations are given in **Table 6.4** in dependency of the position of opening relative to the wind direction.

## Table 6.4: Wind Pressure Distribution Factor $C_i$

| Position of Opening (relative to wind direction) | $C_i$ ** |
|---|---|
| Perpendicular to wind direction, front side of a building | + 0.7 |
| Perpendicular to wind direction, rear side of a building | - 0.5 |
| Parallel to wind direction | - 0.7 |
| Equally distributed on all elevations | + 0.3 |
| Mainly distributed on both front and rear side elevations (perpendicular to wind direction) | - 0.2 |

\*\* = Negative values mean suction

### 6.2.4 Seismic Loads

#### (1) Maximum Credible Earthquake (MCE)

According to the "Earthquake Hazard Atlas - 2 - Egypt" (British Insurance Offices Association, 1990), the probable MMI intensity, with a recurrence period of 50 years, would reach V. This would roughly correlate with a peak horizontal ground acceleration in the range of 0.35 to 0.60 m/sec$^2$ (equivalent to 0.036 to 0.061 g). The Munich Re-Insurance Company arrives at a somewhat lower hazard rating. It assumes a recurrence of intensity VI MMI shaking of 270 years.

Dynamic loading at the site - according to present geotechnic knowledge - could only originate from a (distant) known seismic source or from a floating earthquake from an undefined but potentially existing source. The distance to the faults on the southwest flank of the Red Sea Rift measures about 170 km. The recorded earthquakes are of moderate magnitude, but along a rift a magnitude 8 event cannot be excluded. The peak acceleration at the site would attenuate to about 0.50 m/sec$^2$ (0.051 g).

For comparison, the highest earthquake coefficient given in any of the Egyptian Codes is 0.06 g (specified in the "EC for Soil Mechanics - Part 6", Section 6.2.5.1.1, page 20).

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The effects of the contractual earthquake are to be considered by pseudo-static loads which are to be calculated as follows:

$$- \ P_{gh,s} \ = \ k_{h,s} \ G, \ \text{and} \tag{6.6}$$

$$- \ P_{gv,s} \ = \ k_{v,s} \ G \tag{6.7}$$

Where:

$P_{gh,s} =$ additional force [kN] acting horizontally on each section of a structure (at the centre of gravity of the section), due to the horizontal seismic acceleration during earthquakes;

$P_{gv,s} =$ additional force [kN] acting vertically on each section of a structure (at the centre of gravity of the section), due to the vertical seismic acceleration during earthquakes;

$G \ =$ dead load [kN] of a section of a structure;

$k_{h,s} \ =$ 0.06 (seismic coefficient for MCE in horizontal direction);

$k_{v,s} \ =$ 0.04 (seismic coefficient for MCE in vertical direction).

Based on the "Egyptian Code for Loads and Forces on Structures and Building Works: EC 45/1996" for the Naga Hammadi site (Egyptian earthquake zone II) a seismic coefficient in horizontal direction results in the following figure:

$$k_h = z \cdot i \cdot k \cdot c \cdot s \tag{6.8}$$

Where as:

$z \ = \ $ 0.2 for zone II,

$i \ \leq \ $ 1.25 for important structures,

$k \ \leq \ $ 1.33 for structural system,

$c \ \leq \ $ 0.12 building factor,

$s \ \leq \ $ 1.30 for soil type.

The maximum resulting horizontal acceleration value is $0.2 \cdot 1.25 \cdot 1.33 \cdot 0.12 \cdot 1.3 = 0.052$ g.

For the tender design a value of 0.06 g, as quoted in the EC for Soil Mechanics (see above), was adapted. Thus the design can be considered to be on the safe side regarding seismic considerations.

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## (2) Increase in Water Pressure due to Earthquake

The *increase in water pressure* during seismic events will be calculated by the WESTERGAARD formula, the overall of which is given by:

$$P_{w,s} = 0.583\,k_{h,s} \cdot \gamma_w \cdot H_w^2 \qquad (6.9)$$

Where:

$P_{w,s}$ = overall force, acting horizontally on a submerged structure (per meter in longitudinal direction), due to the increase of water pressure during seismic events, in kN/m;

$k_{h,s}$ = seismic coefficient in horizontal direction, see values for MCE above;

$\gamma_w$ = unit weight of water, in kN/m$^3$;

$H_w$ = head of water at foundation level, in m.

The distribution of the increase of water pressure is given by:

$$p_{w,s}\,(z) = 0.875\,k_{h,s} \cdot \gamma_w \cdot (H_w \cdot z)^{0.5} \qquad (6.10)$$

Where:

$p_{w,s}$ = increase of water pressure, acting horizontally on a submerged structure during seismic events, in kN/m$^2$;

$k_{h,s}$ = seismic coefficient in horizontal direction, see values for MCE above;

$\gamma_w$ = unit weight of water, in kN/m$^3$;

$H_w$ = head of water at foundation level, in m;

$z$ = depth below water table, in m.

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## (3) Increase in Active Earth Pressure due to Earthquake

The *increase in active earth pressure* during seismic events will be considered by calculating the total earth pressure on structures according to the MONONOBE/OKABE formula as follows:

$$P_{ea,s} = 0.5 \cdot (1 + k_{v,s}) \cdot \left( \gamma' \, H_{ea} + \frac{2 \, q \, \sin \alpha_a}{\sin (\alpha_a \, \beta_a)} \right) \cdot H_{ea} \cdot k_{a,s} \qquad (6.11)$$

Where:

$P_{ea,s}$ = total active earth pressure, in kN/m, acting on a retention structure (per meter in longitudinal direction) during seismic events; the inclination measured clockwise from perpendicular to the wall can be assumed as $\delta_a = 2/3 \, \varphi'$ (whereby $\varphi'$ means friction angle of the backfill), with an downward orientation towards the active side of the wall;

$H_{ea}$ = height of backfill behind a retention structure, in m;

$\gamma'$ = submerged unit weight: $\gamma_r - \gamma_w$;

$\gamma_r$ = saturated unit weight of soil, in kN/m³;

$\gamma_w$ = unit weight of water, in kN/m³;

$q$ = specified surcharge, in kN/m²;

$\alpha_a$ = inclination of a retention wall on which active earth pressure is acting (measured clockwise from horizontal);

$\beta_a$ = inclination of the ground surface behind a retention wall (measured clockwise from horizontal);

$k_{a,s}$ = $k_a + 0.75 \, k_{h,s}$ (approximation according to WHITMAN 1990, for dry soil);

$k_a$ = coloumb coefficient for active earth pressure;

$k_{h,s}$ = seismic coefficient in *horizontal* direction, see values for MCE above;

$k_{v,s}$ = seismic coefficient in *vertical* direction, see values for MCE above.

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## 6.3    Stability Analysis

The applicable standard to calculate overall stability is DIN 1054.

The method of calculation of structural stability is compatible with the one given in the Egyptian Code for Soil Mechanics, Design and Construction of Foundation - Part 3: Shallow Foundations, N° 196/1995.

The safety factor against sliding is

$$\eta_S = \frac{H_{f,base} + \Sigma H_{resist}}{\Sigma H_{actio}}$$ (6.12)

where:

$\eta_s$    = safety factor against sliding;

$H_{f,base}$    = horizontal component of friction force acting on the base slab, where $\varphi'$ is taken as specified by the geotechnical report;

$H_{resist}$    = horizontal component of any other resisting force;

$H_{actio}$    = horizontal component of any force acting on a structure.

The safety factor against uplift is

$$\eta_{uplift} = \frac{\Sigma V_{downward}}{\Sigma P_{uplift}}$$ (6.13)

where:

$\eta_{uplift}$    = safety factor against uplift;

$P_{uplift}$    = uplift forces; the resultant of these forces equals the gravity force of the volume of water displaced by a submerged structure;

$V_{downward}$    = permanent vertical downward loads.

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The safety against overturning is achieved if:

- the resultant of all permanent forces acting on the structure above foundation level is acting within the core of the foundation, see **Figure 6.1**;

$$\left(\frac{x_c}{b_x}\right)^2 + \left(\frac{y_c}{b_y}\right)^2 = \frac{1}{9} \qquad (6.14)$$

- the resultant of all forces acting on the structure above foundation level is within an area defined by the following equation (see **Figure 6.2**):
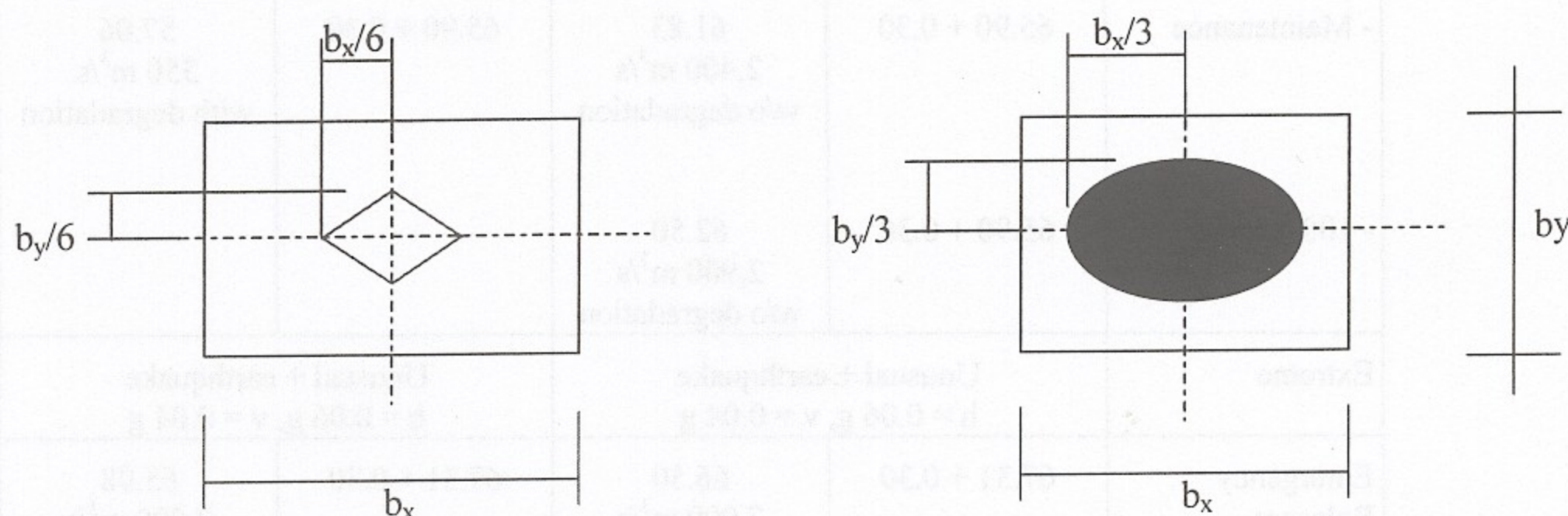


**Figure 6.1: Sketch of Geometric Definitions I - Figure 6.2: Sketch of Geometric Definitions II**

The required factors of safety in accordance with DIN 1045 are as given in **Table 6.5**.

**Table 6.5: Safety Factors**

| Load Case | Seismic Load | | Safety Factors | | Eccentricity of Resultant |
|---|---|---|---|---|---|
| | Horizontal | Vertical | Uplift | Sliding | Overturning |
| Normal | | | 1.10 * | 1.50 | ≤ b/6 |
| Unusual | | | 1.10 * | 1.35 | ≤ b/3 |
| Exceptional | 0.06 g | 0.04 g | 1.05 * | 1.20 | ≤ b/3 |

\* The Egyptian Building Code EC 196/1995 requires a higher safety level against uplift, e.g. in the normal load case 1.20 and in the unusual and exceptional load case 1.10. The stability of the main barrage structures shall also comply with the required factors of safety defined in the EC.

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Stability shall be demonstrated for Load Cases as given in **Table 6.6**.

### Table 6.6: Definition of Load Cases

| Load Case | Uplift | | Sliding | |
|---|---|---|---|---|
| | Headpond U/S Level | Tailwater D/S Level | Headpond U/S Level | Tailwater D/S Level |
| | m.a.s.l. | m.a.s.l. | m.a.s.l. | m.a.s.l. |
| Normal | 65.90 + 0.30 | 61.83 2,400 m$^3$/s w/o degradation | 65.90 + 0.30 | 57.06 350 m$^3$/s with degradation |
| Unusual - Maintenance | 65.90 + 0.30 | 61.83 2,400 m$^3$/s w/o degradation | 65.90 + 0.30 | 57.06 350 m$^3$/s with degradation |
| - 100 Years flood | 65.90 + 0.30 | 62.50 2,900 m$^3$/s w/o degradation | - | - |
| Extreme | Unusual + earthquake h = 0.06 g, v = 0.04 g | | Unusual + earthquake h = 0.06 g, v = 0.04 g | |
| Emergency Releases | 67.31 + 0.30 | 66.50 7,000 m$^3$/s w/o degradation | 67.31 + 0.30 | 65.98 7,000 m$^3$/s with degradation |

### (1)    Maintenance Load Cases (Unusual)

- Sluiceway:          Stoplogs set, bay emptied;
- Powerhouse:          Stoplogs set, bay emptied, generator and turbine runner dismantled;
- Navigation Locks:    Stoplogs set, 1 of 2 chambers emptied.

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## 6.4    Structural Design

### 6.4.1    Construction Material Properties

### (1)    Concrete

Codes applied to concrete mix components and testing are DIN, ASTM, ACI or equivalent Egyptian standards, as approved by the Employer.

The concrete mix design shall comply with the corresponding codes for:

- Aggregate Testing
- Cement Testing
- Cement Replacement Testing, e.g. ASTM and EN
- Concrete Mix Design
- Concrete Testing

Requirements for 4 types of concrete elements considered adequate for the project are given in **Table 6.7**.

### Table 6.7: Concrete Requirements for Individual Elements.

| Item | Elements in contact with water | | Elements not in contact with water | |
|---|---|---|---|---|
| | a) massive t > 0.60 m | b) structural t ≤ 0.60 m | c) massive t > 0.60 m | d) structural t ≤ 0.60 m |
| Characteristic strength* | | | | |
| after 28 days | 22 MPa | 25-30 MPa | 22 MPa | 30MPa |
| after 90 days | 30 MPa | 35 MPa | 30 MPa | - |
| Heat of hydration of cementitious material | less than 50 cal/g (after 7 days) | - | less than 60 cal/g (after 7 days) | - |
| Content of cementitious | less than 400 kg/m$^3$ | - | - | - |

Note *:  The characteristics strength is the compressive strength of the standard cubes (15 x 15 x 15 cm$^3$) below which not more than 5 % of the test results during construction should fall

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## (2) Reinforcement Steel

It is assumed that steel of Egyptian origin is used. Hence, the following specification and gradings (**Table 6.8**) are applicable:

- **ASTM A 615/90** Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
- **ES 262/1988** Steel bars for reinforced concrete
- **ES 1618/1990** Welded steel wire mesh for reinforced concrete
- **ES 76/1969** Metal tension test

### Table 6.8: Reinforcement Steel Properties Definition

| Code | Grade | Yield Strength MPa | Tensile Strength MPa |
|------|-------|--------------------|----------------------|
| ES 262/88 | 24/35 | 240 | 350 |
| ES 262/88 | 40/60 | 400 | 600 |
| ASTM A 615/87 | 60 | 422 | 633 |

## (3) Structural Steel

It is assumed that for limited purpose within the civil works steel of Egyptian origin is used. Hence, the specification and gradings as given in **Table 6.9** are applicable.

### Table 6.9: Structural Steel Properties Definition

| Code | Grade | Yield Strength MPa | Tensile Strength MPa |
|------|-------|--------------------|----------------------|
| ECP 451/1989 | St 37 | 240 | 370 |

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### 6.4.2 Concrete Block Arrangements

#### (1) General

Due to the physical site conditions and foundation requirements of the large size buildings the individual blocks can be subjected to different relative settlements. For this reason the structural systems shall within a certain limitation adapt to the variability of foundation, e.g. by separation of structural elements.

From the constructional point of view and for economic reasons the number of separated blocks shall be minimised. Massive, rigid concrete elements are preferred for hydraulic structures.

The above criteria result in a separation of individual structures with dimensions of up to some 40 m. Structures and individual members within structures will be separated by means of block joints.

#### (2) Block Joints

Block joints are applied, in order to allow for relative displacement but without transferring uncontrolled forces or stresses on to the neighbouring structure or member.

To avoid any relative displacement of the bottom foundation elements, shear keys are introduced. They are designed to transfer normal (vertical and horizontal) forces, meanwhile turning is possible.

The concept is used throughout the main concrete structures, individually adapted to the needs in each structure. The block joints are indicated in the drawings of **Volume 3**, together with shear keys. It should be born in mind that the system of block joints and shear keyes may have to be further adjusted during the construction design as a result of the structural calculations and the actual foundation conditions encountered after construction pit excavation. For construction planning, simultaneous concreting of neighbouring blocks shall be considered as far as possible in order to minimise relative displacements.

### 6.4.3 Applicable Codes and Commentaries

The applicable code for the engineer's design of structures is exclusively:

- **DIN 1045 07.88**            Concrete and Reinforced Concrete – Dimensioning and Implementing

- **DafSt Heft 400 "Supplement 400"**     Commentary to DIN 1045 07.88, Basics of Revision of Limitation of Crack Width

The permissible crack width is given by **Table 6.10**.

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## Table 6.10: Crack Width Limitations

| | Condition for contact with water | Elements deemed to be watertight | Permissible crack width due to load case "heat of hydration" mm | Permissible crack width due to structural loads except axial tension mm |
|---|---|---|---|---|
| Structural elements in contact with water | fluctuating | yes | 0.10 | 0.15 |
| | permanent | yes | 0.10 | 0.15 |
| Massive[1] elements in contact with water | fluctuating | yes | 0.15 | 0.20 |
| | permanent | yes | 0.15 | 0.20 |
| Structural & massive[1] elements in contact with water | fluctuating | no | 0.20 | 0.20 |
| | permanent | no | 0.25 | 0.25 |
| Structural & massive[1] elements **not** in contact with water | dry condition | - | 0.40 | 0.40 |

[1] Note: It is referred to structural elements with a thickness of more than 60 cm as "massive elements"

All values as given above for the allowable crack width are valid under normal conditions. Under transient and extreme conditions (like during earthquakes), the above values for crack width limitations are not applicable.

In order not to exceed the permissible crack widths, the steel stresses have to be restricted according to the requirements as per DIN 1045/Supplement 400.

### 6.4.4   Typical Load Combinations

The critical local combinations to be studied for the individual structures are summarised in **Tables 6.11** to **6.13**.

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## Table 6.11: Load Combinations for Powerhouse (One Pair of Bays)

| | Load Combination | | | | | |
|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI |
| | Headpond & Minimum Tailwater Level | Headpond & 100 Years Flood Tailwater Level | I plus Earthquake | II plus Earthquake | Emergency Release | Final Construction Stage plus Earthquake |
| Headpond Level | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 67.05 + 0.30 | - |
| Tailwater Level | 57.06 | 62.50 | 57.06 | 62.50 | 65.98 | - |
| **Load Cases** | | | | | | |
| Dead Loads* | | | | | | |
| • 100% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| • 94% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| Live Loads** | p | p | p | p | p | p |
| Foundation Load Road | x | x | x | x | x | x |
| Earth Pressure Loads | x | x | x | x | x | x |
| External Hydrostatic Loads | | | | | | |
| • No maintenance (normal) | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance both Bays | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance one Bay of two | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| Crane Loads** | p | p | p | p | p | p |
| Seismic Loads | | | x | x | | x |
| Thermal Loads*,*** | | | | | | |
| • Summer | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |
| • Winter | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |

Note:  *  "$A_1$" means "alternative load cases"

   **  "p" means "live load condition": Investigated are all relevant conditions, including full live load, different load positions and no live load.

   ***  when considering thermal loads, for theses loads the safety factor of 1.75 for the design of the corresponding reinforcement section may be reduced to 1.00.

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## Table 6.12: Load Combinations for Sluiceway (Central Twin Bays)

| | Load Combination | | | | | |
|---|---|---|---|---|---|---|
| | I | II | III | IV | V | VI |
| | Headpond & Minimum Tailwater Level | Headpond & 100 Years Flood Tailwater Level | I plus Earthquake | II plus Earthquake | Emergency Release | Final Construction Stage plus Earthquake |
| Headpond Level | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 67.05 + 0.30 | - |
| Tailwater Level | 57.06 | 62.50 | 57.06 | 62.50 | 65.98 | - |
| **Load Cases** | | | | | | |
| Dead Loads* | | | | | | |
| • 100% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| • 94% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| Live Loads** | p | p | p | p | p | p |
| Foundation Load Road | x | x | x | x | x | x |
| Earth Pressure Loads | x | x | x | x | x | x |
| External Hydrostatic Loads | | | | | | |
| • No Maintenance (normal) | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance Central Bay | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance one Side Bay | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance both Side Bays | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| Crane Loads** | p | p | p | p | p | p |
| Seismic Loads | | | x | x | | x |
| Thermal Loads*,*** | | | | | | |
| • Summer | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |
| • Winter | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |

Note:  *  "$A_1$" means "alternative load cases"

  **  "p" means "live load condition": Investigated are all relevant conditions, including full live load, different load positions and no live load.

  ***  when considering thermal loads, for theses loads the safety factor of 1.75 for the design of the corresponding reinforcement section may be reduced to 1.00.

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### Table 6.13: Load Combinations for Double Navigation Lock (Central Block)

| | Load Combination* | | | | | |
|---|---|---|---|---|---|---|
| | **I** | **II** | **III** | **IV** | **V** | **VI** |
| | Normal Headpond | Normal Headpond | Normal Headpond | critical case out of I to III plus Earthquake | Emergency Release, Lock open/closed | Final Construction Stage plus Earthquake |
| Headpond Level | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 65.90 + 0.30 | 67.05 + 0.30 | - |
| Water Level Left Chamber | 65.90 | 57.06 | 57.06 | 65.09/57.06 | 65.98/65.90 | - |
| Water Level Right Chamber | 57.06 | 65.90 | 57.06 | 65.09/57.06 | 65.90 | |
| **Load Cases** | | | | | | |
| Dead Loads** | | | | | | |
| • 100% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| • 94% | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ | $A_1$ |
| Live Loads *** | p | p | p | p | p | p |
| Collision | p | p | p | - | - | - |
| Foundation Load Road | x | x | x | x | x | x |
| Earth Pressure Loads | x | x | x | x | x | x |
| External Hydrostatic Loads | x | x | x | x | x | - |
| Hydraulic Loads during | | | | | | |
| • Maintenance right Chamber (WL 54.90 m asl) | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| • Maintenance left Chamber (WL 54.90 m asl) | $A_2$ | $A_2$ | $A_2$ | $A_2$ | $A_2$ | - |
| Crane Loads*** | p | p | p | p | p | p |
| Seismic Loads | | | | x | x | x |
| Thermal Loads**, **** | | | | | | |
| • Summer | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |
| • Winter | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ | $A_3$ |

Note:    *    $2^{nd}$ lock (right chamber) closed

     **    "$A_1$" means "alternative load cases"

     ***    "p" means "live load condition": Investigated are all relevant conditions, including full live load, different load positions and no live load.

     **** when considering thermal loads, for theses loads the safety factor of 1.75 for the design of the corresponding reinforcement section may be reduced to 1.00.

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Live loads to be considered on the individual structures are as summarised in **Table 6.14**.

## Table 6.14: Civil Works Live Loads

| Structure | Live Load to be considered (kN/m²) |
|---|---|
| **Powerhouse** | |
| • Access floor elevation 69.00 m asl | 25 |
| • Powerhouse steel cover, elevation 70.00 m asl | 5 |
| • Service bridge, 120 kN truck load, acc. to Egyptian Code 45/1993 | - |
| • Public road, 600 kN truck load, acc. to Egyptian Code 45/199 | - |
| • Tailwater gantry crane beam, 1 crane with concentrated load 2 x P = 250 kN (disturbed over 2.0 m length and spaced 22 m) | - |
| • Main gantry crane beam, 2 cranes beside each other, each crane with concentrated load 2 x P = 470 kN (disturbed over 2.5 m length and spaced 22 m); braking forces shall be considered according to DIN 15018 and DIN 4132 | - |
| • Main floor: elevation 60.55 m asl | 10 |
| • Main floor erection bays: elevation 60.55 m asl | 25 |
| • Floors at all levels, except otherwise noted | 10 |
| • Stairs, gangways, ducts and galleries (not accessible for heavy loads) | 5 |
| • Loads during erection on draft tube slab; elevation 58.20 m asl | 10 |
| • Loads during erection on bottom slab at intake and draft tube | 10 |
| **Sluiceway** | |
| • Main gantry crane beam, 2 cranes beside each other, each crane with concentrated load 2 x P = 470 kN (disturbed over 2.5 m length and spaced 22 m). Braking forces shall be considered according to DIN 15018 and DIN 4132 | - |
| • Machine room slab: elevation 65.50 m asl | 10 |
| • Stairs | 5 |
| • Service bridge, 120 kN truck load, acc. to Egyptian Code 45/1993 | - |
| • Public road, 600 kN truck load, acc. to Egyptian Code 45/1993 | - |
| • Loads during erection on bottom slab | 10 |
| **Navigation Lock** | |
| • Access floor: elevation 69.00 m asl | 15 |
| • Gantry crane, 1 crane with concentrated load 1 x P = 400 kN (distributed over 3 m length). Braking forces shall be considered according to DIN 15018 and DIN 4132 | - |
| • Room slabs below elevation 69.00 m asl | 10 |
| • Loads during erection on bottom slab of chamber | 15 |

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### 6.4.5  Minimum Reinforcement Requirements

**(1)  General**

    a)   Need for Minimum Reinforcement

      -   Influence of the concrete mix design on restraint effects;

      -   Influence of the length of a structural member on the required amount of minimum reinforcement;

      -   Influence of the thickness of a structural member on the required amount of minimum reinforcement.

    b)   Acceptable Crack Width

    c)   Design for Restraint Effects due to Heat of Hydration

      -   Horizontal direction;

      -   vertical direction.

    d)   Design for Restraint Effects due to Service Temperature Gradients

      -   Horizontal direction;

      -   vertical direction.

    e)   Proposal for the Minimum Reinforcement

      -   Massive elements deemed to be watertight (crack width 0.15 mm);

      -   massive elements deemed to be waterproof (crack width 0.20 mm);

      -   massive elements not deemed to be watertight (crack width 0.25 mm).

    f)   Effect of Restraint and Structural Loads and on Reinforcement Requirements

    g)   Conclusions

      -   Literature/Basic Documents

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## (2)  Need for Minimum Reinforcement

Restraint effects are stresses caused by impeded deformation in a structural element, which can either result from the effect of support (e.g. from differential settlements, previous concreting stages, causing restraint in early concrete) or from parts of the element itself (e.g. strain due heat of hydration or service temperature). Restraint effects occur independent from structural loads, and the stresses induced by restraint decrease when the impediment to deformations is relieved, e.g. by formation of cracks.

The restraint in a structural system depends on its degree of structural in-determination, which can be influenced by design, e.g. by arrangement of expansion (contraction) joints. If this is not feasible, and the concrete members will be exposed to restraint and related tensile stresses which exceed the tensile strength of concrete, cracks will develop. Steel reinforcement is needed to limit the crack width by distributing a few large cracks into a larger number of small cracks of acceptable width.

The amount of reinforcement steel needed to achieve adequate crack width and distribution of cracks is referred to as "minimum reinforcement", which is independent from results of the structural calculations by which the effects resulting from structural loads are considered. Especially, in case of massive concrete members, restraint in early concrete can considerably influence the reinforcement design. The heat generated during hydration of the cementitious materials contained in the concrete is the determining factor for the amount of minimum reinforcement.

### (2.1)  Concrete Mix Design effecting Restraint

When a concrete element hardens and thereby warms up by heat from hydration, restraint induces compressive stresses only. Because of the ability of the concrete to withstand compression, the design is not influenced by the restraint from this period. During the period thereafter, heat dissipates to the environment, which finally leads to a temperature equilibrium in the surface near zones of the element. Depending on the boundary conditions of the concrete element and the corresponding restraint characteristics, this can induce either tensile or compressive stresses. If the stress level exceeds the tensile strength of concrete, cracks will form. The required amount of minimum reinforcement by which crack distribution results in a sufficient number of small cracks of acceptable width, depends on the expected change of volume, after the temperature peak caused by hydration has passed.

For massive concrete members, the temperature increase resulting from hydration is normally higher than for thin members. Hence, the probability of crack formation in massive concrete members is higher, and measures to control and reduce the heat of hydration and the related volume change should be applied, i.e. by:

- Application of cement with low heat generation,

- replacement of a larger percentage of the cement by fly ash and highblast furnace slag, which are less reactive;

- reducing the total quantity of cementitious materials in the concrete mix;

- limitation of the temperature of the fresh concrete at the time of placement, which also influences the rate of temperature rise in a concrete member, by cooling of the concrete mix;

- temperature insulation to control the rate of heat absorbed or lost.

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## (2.2) Structural Member Length effecting the Minimum Reinforcement

For massive structures with large dimensions, restraint due to heat of hydration results mainly from:

- non-linear temperature gradient in the thick outer walls and base slabs, and
- casting against concrete poured earlier during construction.

Due to the non-linear temperature gradient, tensile stresses build-up in areas near the surface , and compressive stresses in the inner areas. This part of the phenomenon is the so called "self-equilibrating" stress component. The remaining stress component is called "axial restraint", and induces tensile stresses over the entire cross section. The resulting normal stresses depend on the geometry of the casting stages.

In order to effectively reduce the axial restraint, the length of the concreting stage must be less than two times the height. Assuming the typical height of concrete lifts by 3 to 4 meters, the length of a concrete pour must be shorter than the corresponding critical length of 6 to 8 meters. This would require a large number of joints which constitute weak points in a water retaining structure.

If the length of a casting stage is much larger than the above mentioned critical length (i.e. length/height of pour = 10 or larger), then the temperature distribution due to heat of hydration does not change over the length, and the boundary influence from the distant ends of the cast is not notable in the central sections. Hence, restraint induced tensile stresses remain almost constant over the height of the considered concrete lift, and the minimum reinforcement has to be applied over the full height of the pour.

## (2.3) Structural Member Thickness effecting Minimum Reinforcement

According to [14], the influence of the structural member thickness on the amount of minimum reinforcement is limited to 1.60 m. An exception from this rule are concrete elements of much larger dimensions, as e.g. foundation slabs, which during cooling following the heat peak of hydration, are exposed to restraint from friction with the ground. In these cases, higher minimum reinforcement than corresponding to a wall thickness of 1.60 m may has to be considered.

## (3) Acceptable Crack Width

For elements exposed to water pressure on the outer side, and which are deemed to be watertight, their service ability depends strongly on the width of cracks. Waterproofness of concrete structures during the service life can be guaranteed by limiting the crack width to an acceptable target, i.e. by reduction of the heat of hydration and by reinforcement. With the development of restraint, the different load combinations, and the large dimensions of massive hydraulic structures, a complete prevention of cracks is normally not possible.

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The acceptable crack width depends mainly on the following parameters:

- the required degree of the impermeability (e.g. higher for computer rooms, lower for inspection galleries);

- the ratio between hydrostatic pressure (h), measured in equivalent height of water column, and the thickness (d) of the concrete element (h/d = hydraulic gradient); and

- the self-healing ability of cracks.

For the calculation of the width of cracks developing in the outer members of a water retaining structure, the effects of:

- heat generation of cement taking into account the pouring sequence,

- and the prevailing temperature differences and the corresponding load cases

are the fundamental parameters to be defined.

In any case, the uncertainty of the model applied to crack width calculation has to be taken into consideration. The uncertainty increases with the thickness of the structural members and with the degree of limiting the crack width. Already small crack widths can only be further reduced by a large quantity of additional reinforcement.

Theories applied for crack width calculation (DIN 1045/Supplement 400, EC2, ACI and MC90) are based on the behaviour of a tension tie, with a uniformly reinforced cross section. Actually, to achieve structural efficiency of reinforcement in thick concrete members, the steel is placed non-uniformly distributed in the cross section, e.g. concentrated near the outer faces of concrete. Another effect, that of bond deterioration in the vicinity of a crack, increases with the size of bar diameter. Considering these and other uncertainties of the theoretical approach, it appears not to be reasonable to require a calculatory crack width smaller than 0.1 mm. For massive concrete members, as in the case of the project, the reasonable limit of crack width should not be less than 0.15 mm.

A smaller calculation crack width would lead to considerably more reinforcement at the faces of massive walls, impeding proper placement and compaction of fresh concrete. As a result, gravel pockets, and voids in the surface area of the concrete member may form, especially in congested areas, e.g. at the connections of frames or beams. This may increase the sensitivity against corrosion of the reinforcement, and may reduce the permeability of the structural members considerably. Therefore, concentration of large amounts of reinforcement near the surface of the concrete walls exposed to water should be avoided.

Experience with comparable types of structures has shown that a calculatory crack width of 0.15 mm suffices to guarantee the service ability of the structure. By recent investigations [12], calculatory crack widths of 0.15 mm are proposed for a hydraulic gradient h/d ≤ 20. At this water pressure gradient, permanent leakage is impeded by self-healing effects, a common phenomenon contributing to the reliability of structures exposed to external water pressure. A calculatory crack width of 0.15 mm is considered to be the appropriate for massive outer walls of hydraulic structures which are deemed to be watertight.

Experience with restraint effects in massive concrete members has underlined the importance for a safe estimation of the internal force to be borne by the reinforcement, in order to obtain realistic results for the minimum reinforcement required for limiting cracks to a certain calculatory width.

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## (4)   Design for Restraint Effects due to Heat of Hydration

### (4.1)   Horizontal Direction

The temperature changes due to dissipation of the heat from hydration govern the development of the restraining force in horizontal direction. The restraining force in the outer walls can be estimated in accordance with DIN 1045/Supplement 400 [9], as follows:

$$F = A_c \cdot k_t \cdot f_{ct,ef} \cdot k_e \cdot k \qquad (6.15)$$

where:

$A_c$ = area of the concrete cross section,

$k_t$ = factor considering the time, at which the crack due to dissipation of heat will form (if the crack develops 28 days after placing of concrete, then $k_t = 1.0$ has to be assumed),

$f_{ct,ef}$ = tensile strength of concrete after 28 days,

$k_e$ = factor considering the self-equilibrating stresses due to the non-linear temperature profile within the cross section,

$k$ = factor considering the stress distribution within the section, immediately prior to cracking. This factor considers the type of restraint, and hence the interaction between different stages of concreting.

F is the restraining force, which has to be borne by reinforcement. The amount of reinforcement required for controlling the restraint in horizontal direction depends on the calculatory crack width. From both, restraining force and selected calculatory crack width, the required amount of reinforcement can be calculated applying the formulae given in [9].

### (4.2)   Vertical Direction

Since volume change due to temperature rise during hardening of concrete is not impeded, no restraint is induced in vertical direction of massive walls.

## (5)   Design for Restraint Effects due to Service Temperature Gradients

### (5.1)   Horizontal Direction

Once constructed and taken into service, there remain no restraint effects requiring more than the minimum reinforcement which resulted from volume change due to heat of hydration. Considering the effects of long term loading on the tensile strength of concrete, the cracking force in the outer walls under service conditions corresponds to the restraining force given by formula (6.15); larger forces are not expected. On the other side, during service the sensibility of the structure to restraint effects (volume changes, settlements) is reduced, because crack formation during construction leads to stiffness reduction of the structural elements.

### (5.2)   Vertical Direction

The effect of changing temperature gradients during the service period can be important for massive walls. E.g., by contact with water on one side, the temperature gradient will provoke bending with tensile stresses on the one and compressive stresses on the opposite side.

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It is assumed that the maximum absorbable bending moment corresponds to the cracking moment of the structural member, e.g.:

$$M_{cr} = 0.8 \, f_{ct,ef} \, \frac{d^2 \, b}{6} = M_{max} \qquad (6.16)$$

where:

| | | |
|---|---|---|
| 0.8 | = | factor considering the effect of long term loading on the tensile strength of concrete |
| d | = | thickness of the member |
| b | = | width of the member |
| $f_{ct,ef}$ | = | tensile strength of concrete after 28 days |

The reinforcement of the outer walls in vertical direction has to be sufficient to withstand the tension forces from the cracking moment as given above. The effect of the longitudinal compressive force in the walls can be neglected.

## (6)    Proposed Minimum Reinforcement

### (6.1)    Massive Elements deemed to be Watertight (Crack Width 0.15 mm)

The minimum reinforcement as given in the tables below is calculated for a low heat concrete mix (corresponding to 300 kg/m$^3$ German "Aquadur" cement) and for a restraining force calculated in accordance with formula (6.15) and DIN 1045/Supplement 400 [9].

#### Table 6.15:  Minimum Reinforcement (Crack Width 0.15 mm)

| Ø [mm] – Spacing [cm] | Longitudinal Direction | | Lateral/Vertical Direction | |
|---|---|---|---|---|
| | 1st Layer | 3rd Layer | 2nd Layer | 4th Layer |
| **Base Slab (d ≥ 4.00 m, e.g. Powerhouse)** | | | | |
| Outer side (wet) | Ø 28 – 15 | Ø 28 – 15 | Ø 28 – 15 | Ø 28 – 15 |
| Inner side (dry) | Ø 28 – 15 | Ø 28 – 15 | Ø 28 – 15 | Ø 28 – 15 |
| **Walls, d ≥ 1.60 m** | | | | |
| Outer side (wet) | Ø 25 – 15 | Ø 25 – 15 | Ø 25 – 15 | Ø 22 – 30 |
| Inner side (dry) | Ø 25 – 15 | Ø 25 – 15 | Ø 25 – 15 | – |
| **Walls, 1.00 m < d ≤ 1.60 m** | | | | |
| Outer side (wet) | Ø 22 – 15 | Ø 22 – 15 | Ø 20 – 15 | Ø 20 – 30 |
| Inner side (dry) | Ø 22 – 15 | Ø 22 – 15 | Ø 20 – 15 | – |
| **Walls, d ≤ 1.00 m** | | | | |
| Outer side (wet) | Ø 25 – 15 | – | Ø 20 – 15 | – |
| Inner side (dry) | Ø 25 – 15 | – | Ø 20 – 15 | – |

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## (6.2) Massive Elements deemed to be Waterproof (Crack Width 0.20 mm)

For concrete elements located within the range of fluctuating water levels, which are not deemed to be watertight, a crack width limitation to 0.2 mm (calculated in accordance with Supplement 400 [9]) is regarded as sufficient. Hence, the minimum reinforcement given by the **Table 6.16** is applicable.

### Table 6.16: Minimum Reinforcement (Crack Width 0.20 mm)

| Ø [mm] – Spacing [cm] | Longitudinal Direction | | Lateral/Vertical Direction | |
|---|---|---|---|---|
| | 1st Layer | 3rd Layer | 2nd Layer | 4th Layer |
| **Base Slab (d ≥ 3.00 m, e.g. Powerhouse)** | | | | |
| Outer side (wet) | Ø 22 – 15 | Ø 22 – 15 | Ø 22 – 15 | Ø 22 – 15 |
| Inner side (dry) | Ø 22 – 15 | Ø 22 – 15 | Ø 22 – 15 | Ø 22 – 15 |
| **Walls, d ≥ 1.60 m** | | | | |
| Outer side (wet) | Ø 20 – 15 | Ø 20 – 15 | Ø 20 – 15 | Ø 20 – 30 |
| Inner side (dry) | Ø 20 – 15 | Ø 20 – 15 | Ø 20 – 15 | |
| **Walls, 1.00 m < d ≤ 1.60 m** | | | | |
| Outer side (wet) | Ø 25 – 15 | – | Ø 20 – 15 | – |
| Inner side (dry) | Ø 25 – 15 | | Ø 16 – 15 | – |
| **Walls, d ≤ 1.00 m** | | | | |
| Outer side (wet) | Ø 20 – 15 | | Ø 16 – 15 | |
| Inner side (dry) | Ø 20 – 15 | | Ø 16 – 15 | |

## (6.3) Massive Elements not deemed to be watertight (crack width 0.25 mm)

For concrete elements subjected to the condition of high humidity, a permissible crack width of 0.25 mm (calculated in accordance with Supplement 400 [9]) is considered to be adequate. The same crack width criteria may be applied for elements in permanent contact with water, which are not deemed to be watertight. For these cases, the minimum reinforcement as given in **Table 6.17** is applicable.

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### Table 6.17:  Minimum Reinforcement (Crack Width 0.25 mm)

| Ø [mm] – Spacing [cm] | Longitudinal Direction | | Lateral/Vertical Direction | |
|---|---|---|---|---|
| | 1st Layer | 3rd Layer | 2nd Layer | 4th Layer |
| **Base Slab (d ≥ 4.00 m)** | | | | |
| Outer & inner side | Ø 22 – 15 | Ø 22 – 30 | Ø 22 – 15 | Ø 22 – 30 |
| **Walls, d ≥ 1.60 m** | | | | |
| Outer & inner side | Ø 22 – 15 | – | Ø 20 – 15 | – |
| **Walls, 1.00 m < d ≤ 1.60 m** | | | | |
| Outer & inner side | Ø 20 – 15 | – | Ø 16 – 15 | – |
| **Walls, d ≤ 1.00 m** | | | | |
| Outer & inner side | Ø 16 – 15 | – | Ø 14 – 15 | – |

The influence of the crack width on the service ability and durability of the construction is considerably smaller as in the case of elements deemed to be watertight. According to DIN 1045, a crack width of 0.4 mm is permissible in dry conditions (small or negligible hazard for corrosion). The corresponding minimum reinforcement is negligible in case of hydraulic structures, where due to constructive considerations larger re-bar diameters are applied.

### (7)    Effect of Restraint and Structural Loads on Total Reinforcement

The above estimate of the required reinforcement was performed with the assumption, that the reinforcement can bear the cracking force of the concrete element under deformation within the permissible crack width. In a reinforced concrete element exposed to restraint, the restraining force remains at the level of the cracking as long as new single cracks can form in the concrete. Any increase of volume is compensated by formation of further cracks. Thus, the theory of elasticity which assumes that the restraining force increases with volume change, is not applicable to the case, when ongoing formation of cracks result in a reduction of the internal forces in the concrete element. The restraint force only increases after the crack pattern has stabilised. The increase of the restraining force corresponds to the overall stiffness of the cracked element, which is considerably smaller than the stiffness of the uncracked state. This is shown by **Figure 6.3** in the following graphic.
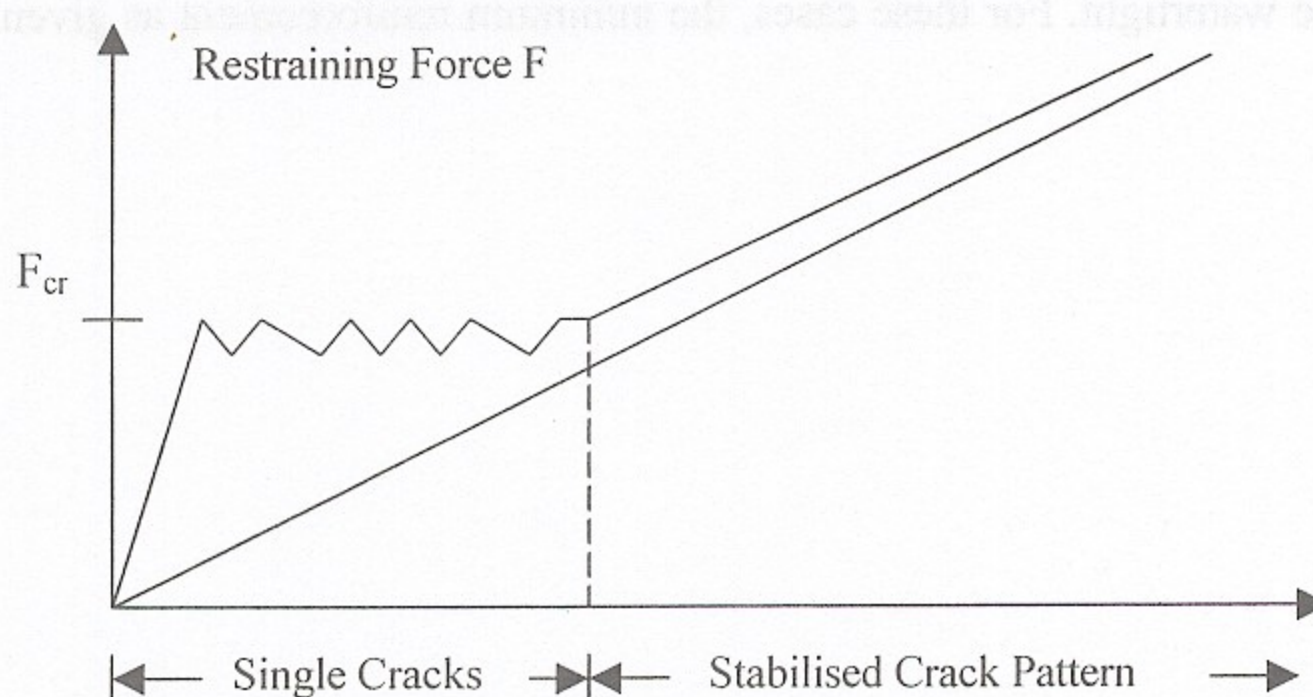


Figure 6.3: Restraining Force versus Volume Change in Reinforced Concrete Elements [8]

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For normal applications of mass concrete in hydraulic structures, the volume change due to dissipation of hydration heat does not exceed the range of single crack formation, as shown on the above graph. The single crack range usually extends to a strain of about 0.8‰, corresponding to a temperature differential of 80 °C, (which is much higher than what has to be expected). Crack formation during the service period by structural loads, together with creeping of concrete, leads to a reduction of the restraining forces, which are resulting from heat generation in early age of concrete and from temperature difference during service period.

For the above reasons, in many typical applications of massive hydraulic structures, the minimum reinforcement in both directions can be taken into account for the required reinforcement due to structural loads.

## (8)   Conclusions

Particularly in massive concrete elements, from which most of the hydraulic structures are composed, dissipation of the heat of hydration provides restraining forces which have to be taken into consideration for the design. In order to limit the calculatory crack width to a certain value, a corresponding minimum reinforcement has to be applied in both directions. It can be taken into account for the reinforcement required by the structural loads. As in hydraulic structures the amount of reinforcement required by structural loads is frequently less than the reinforcement needed for the restraint effects, only the minimum reinforcement is applied.

### 6.4.6   Sample Analyses

Sample analyses are given for the powerhouse, the sluiceway, and the navigation lock in the paragraphs below. Further the contents of future structural calculation notes is indicated in a separate paragraph.

### (1)   Powerhouse

-   **Structural Model:** The structural computation for the powerhouse is based on a 3D finite element model, by which a pair of two bays was modelled. Spring elements were introduced for modelling the subgrade reaction (modulus taken as 10 MN/m$^3$) between base slab and foundation ground. By this method the complex geometry of the powerhouse was modelled with some geometric simplifications.

    The selected method comprises both

    -   calculation of the stresses acting on each element, and
    -   analysis of the strain to which each individual element is exposed.

    For the raft, a separate model based on shell elements was established, to which loads were applied as resulted from the stability calculations, see **Appendix 6.1**. For the implementation of the 3D finite element models, SAP90 software was applied.

-   **Load Cases and Load Combinations:** It is referred to **Table 6.11, Section 6.4.4**.

- **Limitation of Crack Width:** A crack width limitation to 0.15 mm (DIN 1045/Supplement 400) in accordance with the criteria as given by **Table 6.10** is applied for the outer walls of the machine hall and for other elements which are deemed to be watertight. A calculatory crack width of 0.20 mm is applied for the members in contact with water which are not deemed to be watertight, and a crack width of 0.25 mm is applied for members not in contact with water.

For limitation of cracking due to restraint effects, a minimum reinforcement was determined with consideration of all relevant effects (heat of hydration, construction stages, thermal loads after construction, etc.), as described in **Section 6.4.5**.

- **Method of Cross Checking:** For cross checking of the sectional reinforcement design, partial 2D slab and frame models were applied.

### (2) Sluiceway

- **Structural Model:** The structural computation for the Sluiceway is based on a 2D frame model, (SAP90 software) by which a typical cross section of one bay was modelled. Spring elements for modelling of the subgrade reaction were introduced as for (1).

- **Load Cases and Load Combinations:** It is referred to **Table 6.12, Section 6.4.4**.

- **Limitation of Crack Width:** A crack width limitation to 0.20 mm (DIN 1045/Supplement 400) is considered, in accordance with the criteria given by **Table 6.10**.

For limitation of cracking due to restraint effects, a minimum reinforcement was determined with consideration of all relevant effects (heat of hydration, construction stages, thermal loads after construction, etc.), as described in **Section 6.4.5**.

- **Method of Cross Checking:** The sectional and reinforcement design was cross checked by manual calculations.

### (3) Navigation Lock

- **Structural Model:** same as for (1) and (2)

- **Load Cases and Load Combinations:** It is referred to **Table 6.13, Section 6.4.4**.

- **Limitation of Crack Width:** Criteria are applied as set out in **Section 6.4.5**.

- **Method of Cross Checking:** The sectional and reinforcement design was cross checked by manual calculations.

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**(4)   Contents of Future Structural Calculation Notes**

The structural design of each structure during the construction design phase shall be documented by structural calculation notes with approximately the following contents:

- Introduction
- Geometry / system description
- Applied standards and commentaries
- Construction material properties
- Load cases
- Load combinations
- Structural model / analysis
- Results
- Reinforcement sketches
- Method of cross checking
- Conclusion

### 6.4.7   Literature/Basic Documents

[9]   Grundlagen der Neuregelung zur Beschränkung der Rißbreite by Peter Schießl in Heft 400 of the Deutschen Ausschuß für Stahlbeton (Supplement 400)

[10]   Beton - Arten-Herstellung-Eigenschaften, Helmut Weigler, Sieghart Karl,   Verlag Ernst & Sohn, Berlin

[11]   Grundlagen und Bemessungshilfen für die Rißbreitenbeschränkung im Stahlbeton und Spannbeton by Gert König and Nguyen Viet Tue in Heft 466 of the Deutschen Ausschuß für Stahlbeton (Supplement 466)

[12]   Wasserdurchlässigkeit und Selbstheilung von Trennrissen in Beton by Carola Katharina Edvardsen in Heft 455 of the Deutschen Ausschuß für Stahlbeton (Supplement 455)

[13]   Rißbreitenbeschränkung zwangbeanspruchter Bauteile aus hochfestem Normalbeton by Harald Bergner im Heft 482 of the Deutschen Ausschuß für Stahlbeton (Supplement 482)

[14]   Rißbreitenbeschränkung nach DIN 1045: Diagramme zur direkten Bemessung by Günter Meyer, Beton-Verlag, Düsseldorf (1989)

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# APPENDICES

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# Appendix 6.1

# Stability Calculations

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## CALCULATION SHEETS

- Navigation Lock

- Sluiceway

- Powerhouse

- Powerhouse Abutment Pier

- Powerhouse U/S Abutment Pier

- Powerhouse D/S Abutment Pier

**Stability Calculation**
for
**NAVIGATION LOCK**

*CALCULATION SHEET*

Final Calculation August 1999

# CALCULATION SHEET

Subject : **NAVIGATION LOCK** (Summary Sheet)

| Load Case | | Item | Driving Force | Resisting Force | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|
| Normal | | Sliding | 115.3 | 229.2 | **1.99** | 1.50 |
| Normal | | Uplift | 821.6 | 1181.5 | **1.44** | 1.10 |
| Unusual | Maintenance | Sliding | 115.3 | 205.9 | **1.78** | 1.35 |
| Unusual | Maintenance | Uplift | 821.6 | 1144.8 | **1.39** | 1.10 |
| Unusual | 100 years flood | Sliding | N.A. | N.A. | **N.A.** | 1.35 |
| Unusual | 100 years flood | Uplift | N.A. | N.A. | **N.A.** | 1.10 |
| Extreme | | Sliding | 164.9 | 198.9 | **1.21** | 1.20 |
| Extreme | | Uplift | 821.6 | 1099.0 | 1.34 | 1.05 |
| Emergency | | Sliding | 108.1 | 409.7 | **3.79** | 1.20 |
| Emergency | | Uplift | 896.8 | 1540.0 | **1.72** | 1.05 |

Remarks:
* N.A. = Not Applicable

| Prepared / Date | Revision |
|---|---|
| Checked / Date | Sheet      of |

# Naga Hammadi Barrage Development Consultant

## CALCULATION SHEET

Subject : **Stability Calculation for NAVIGATION LOCK**

Remarks :

* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)

* Load cases according to Design Criteria Memorandum Draft Version : **27.07.99**

* Safety factors according to DIN 1054.

## 1  General

### 1.1  Input Data

| | | | | | |
|---|---|---|---|---|---|
| Coefficient of friction (Cf) | = | 0.637 | Coefficient of earth pres. (k0) | = | 0.463 |
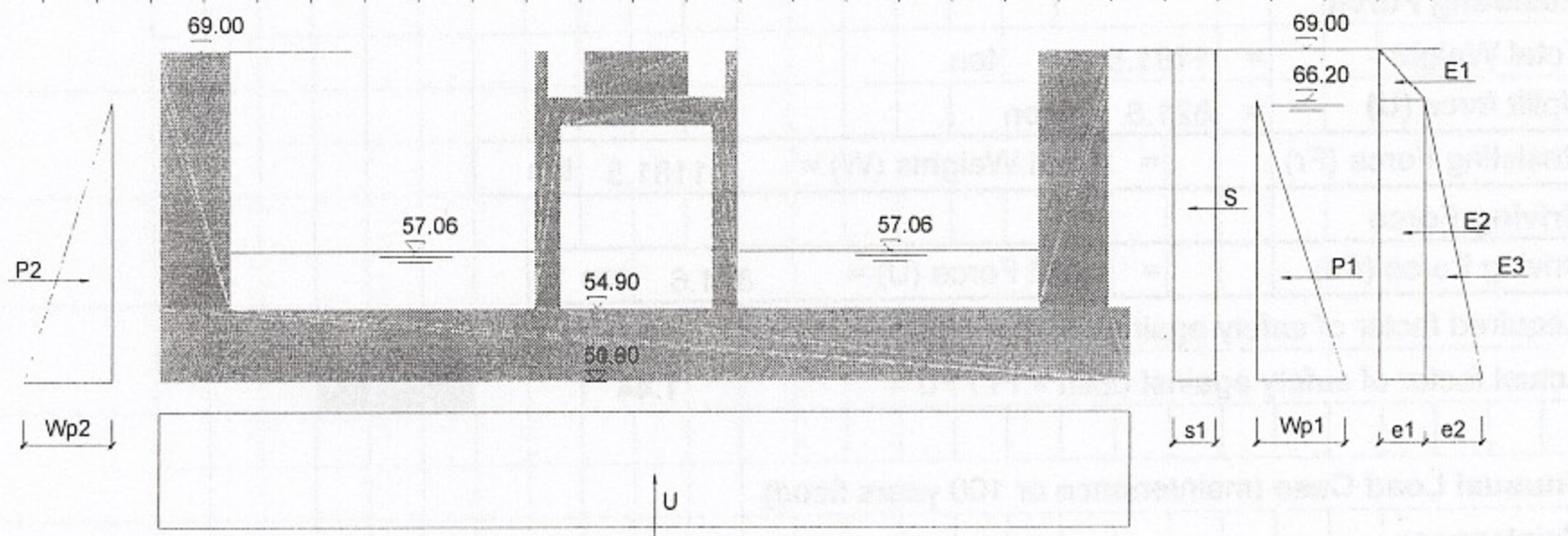


### 1.2  Structure Geometry

| | | | | | | |
|---|---|---|---|---|---|---|
| Foundation Level | = | 50.90 | masl | | | |
| B = 1.00 m | qf = | 1.00 | | q = | 2.0 | ton/m$^2$ |
| Concrete weight | = | 955.0 | ton | | | |
| Sandfill Weight | = | 153.0 | ton | | | |
| Total Weight | = | 1108.0 | ton | | | |

## 2  Load Cases

### 2.1  Normal Load Case

#### 2.1.1  Sliding

| | | | |
|---|---|---|---|
| Water Level | = | 66.20 | masl |
| Water Weight | = 73.5 | ton | |
| Total Wight (W) | = 1181.5 | ton | |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| e1 | = | 2.33 | ton/m$^2$ | e2 | = | 7.79 | ton/m$^2$ |
| Wp1 | = | 15.30 | ton/m$^2$ | s1 | = | 0.93 | ton/m$^2$ |
| Wp2 | = | 15.30 | ton/m$^2$ | | | | |
| E1 | = | 3.27 | ton | E2 | = | 35.70 | ton |
| E3 | = | 59.61 | ton | S | = | 16.76 | ton/m$^2$ |
| P1 | = | 117.0 | ton | P2 | = | 117.0 | ton |

a) **Resisting Force**

| | | | | | | |
|---|---|---|---|---|---|---|
| Uplift force (U) | = | 821.6 | ton | | | |
| Sum of vertical force (V) | = | (W) - (U) | = | 359.9 | ton | |
| Resisting Force (Fr) | = | (V)x(Cf) | = | 229.2 | ton | |

| | |
|---|---|
| Prepared / Date | Revision |
| Checked / Date | Sheet      of |

Subject : **Stability Calculation for NAVIGATION LOCK**

| | | | | | |
|---|---|---|---|---|---|
| b) | **Driving Force** | | | | |
| | Driving Force (Fd) | = | E1+E2+E3+S+P1-P2 | = | 115.3 ton |
| | Required factor of safety against sliding = | | | | 1.50 |
| | Actual factor of safety against sliding = Fr / Fd = | | | | **1.99** Safe |

**2.1.2 Uplift**

Water Level = 66.20 masl

a) **Resisting Force**

Total Weight = 1181.5 ton

Uplift force (U) = 821.6 ton

Resisting Force (Fr) = Total Weights (W) = 1181.5 ton

b) **Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) = | 821.6 ton |
| Required factor of safety against uplift = | | | 1.1 |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.44** Safe |

**2.2 Unusual Load Case** (maintenance or 100 years flood)

**2.2.1 Maintenance**

**2.2.1.1 Sliding**

Water weight = 36.8 ton (one empty)

Total Weight = 1144.8 ton

Uplift force (U) = 821.6 ton

Sum of vertical force (V) = (W) - (U) = 323.2 ton

Resisting Force (Fr) = (V) x (Cf) = 205.9 ton

| | | | | |
|---|---|---|---|---|
| Driving Force (Fd) | = | E1+E2+E3+(P1-P2)+S | = | 115.3 ton |
| Required factor of safety against sliding = | | | | 1.35 |
| Actual factor of safety against sliding = Fr / Fd = | | | | 1.78 Safe |

**2.2.1.2 Uplift**

a) **Resisting Force**

Resisting Force (Fr) = Total Weights (W) = 1144.8 ton

b) **Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) = | 821.6 ton |
| Required factor of safety against uplift = | | | 1.1 |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.39** Safe |

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|---|---|
| Checked / Date | Sheet          of |

| 2.2.1 | 100 years flood |
|---|---|
| | not applicable |

**2.3 Extreme Case (Maintenance + earthquake)**

**2.3.1 Sliding**

Water Level U/S = 66.20 masl

$e_h$ = 0.06    H = 15.30

$\gamma$ = 1 ton/m$^3$    B = 1.0 m

Seismic horiz. force due to water pres. $(H_{we})$ = $0.583 \times h^2 \times e_h \times B$

= 8.19 ton

Sesmic factor for horizontal load $(e_h)$ = 0.06

Sesmic factor for vertical load $(e_v)$ = 0.04

Horiz. Force due to dead load $(H_{Ge}) = (e_h) \times (W)$ = 70.9 ton

Vert. Force due to dead load $(V_{Ge}) = (e_v) \times (W)$ = 45.8 ton

Seiemic force for Earth Pressure = $0.5 (1+k_{v,s}) (\gamma H + 2q.\sin\alpha/\sin(\alpha+\beta)) H k_{a,s}$

$E_{ea}$ = 77.59 ton

Seiemic force for Earth Pressure = $0.5 (1+k_{v,s}) \gamma H^2 k_{p,s}$

$E_{ep}$ = 22.24 ton

**a) Resisting Force**

Sum of vertical force $(Ve) = (V) - (V_{Ge})$ = 277.4 ton

Resisting Force $(Fr)$ = $(Ve) \times (Cf) + E_{ep}$ = 198.9 ton

**b) Driving Force**

Driving Force $(Fd) = H_{ge} + H_{we} + Es$ = 164.9 ton

Required factor of safety against sliding = 1.20

Actual factor of safety against sliding = Fr / Fd = **1.21**     Safe

**2.3.2 Uplift**

**a) Resisting Force**

Resisting Force $(Fr) = (W) - (V_{Ge})$ = 1099.0 ton

**b) Driving Force**

Driving Force $(Fd)$ = Uplift force $(U)$ = 821.6 ton

Required factor of safety against uplift = 1.05

Actual factor of safety against uplift = Fr / Fd = **1.34**     Safe

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| Checked / Date | Sheet        of |

## CALCULATION SHEET

Subject :   **Stability Calculation for NAVIGATION LOCK**

| | | | | | | |
|---|---|---|---|---|---|---|

**2.4**   **Emergency Releases**

**2.4.1**   **Sliding**

Water Level U/S   =   67.60   masl

a)   **Resisting Force**

water weight =   432.0   ton

Total weight   =   1540   ton

Uplift Force (U) =   896.79   ton

Sum of vertical force (V)   =   643.2   ton

| | | | | | |
|---|---|---|---|---|---|
| $e_1$ = | 1.17 | ton/m$^2$ | $e_2$ = | 8.51 | ton/m$^2$ |
| $W_{p1}$ = | 16.70 | ton/m$^2$ | $s_1$ = | 0.93 | ton/m$^2$ |
| $W_{p2}$ = | 16.70 | ton/m$^2$ | | | |
| $E_1$ = | 0.82 | ton | $E_2$ = | 19.48 | ton |
| $E_3$ = | 71.02 | ton | $S$ = | 16.76 | ton/m$^2$ |
| $P_1$ = | 139.4 | ton | $P_2$ = | 139.4 | ton |

Resisting Force (Fr)   =   (V) x (Cf)   =   409.7   ton

b)   **Driving Force**

Driving Force (Fd)   =   E1+E2+E3+S+(P1-P2)   =   108.1   ton

Required factor of safety against sliding =   1.20

Actual factor of safety against sliding = Fr / Fd =   **3.79**   Safe

**2.4.2**   **Uplift**

a)   **Resisting Force**

Water Level U/S   =   67.60   masl

a)   **Resisting Force**

Total weight   =   1540   ton

Resisting Force (Fr)   =   Total Weights (W) =   1540.0   ton

b)   **Driving Force**

Driving Force (Fd)   =   Uplift force (U) =   896.8   ton

Required factor of safety against uplift =   1.10

Actual factor of safety against uplift = Fr / Fd =   **1.72**   Safe

| | |
|---|---|
| Prepared / Date | Revision |
| Checked / Date | Sheet        of |

# Stability Calculation
## for
# SLUICEWAY

## *CALCULATION SHEET*

Final Calculation August 1999

# CALCULATION SHEET

Subject : **SLUICEWAY** (Summary Sheet)

| Load Case | | Item | Driving Force | Resisting Force | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|
| Normal | | Sliding | 3767.8 | 10063.5 | 2.67 | 1.50 |
| Normal | | Uplift | 23150.0 | 36850.0 | 1.59 | 1.10 |
| Unusual | Maintenance | Sliding | 3767.8 | 9175.5 | 2.44 | 1.35 |
| Unusual | Maintenance | Uplift | 23150.0 | 34750.0 | 1.50 | 1.10 |
| Unusual | 100 years flood | Sliding | N.A. | N.A. | N.A. | 1.35 |
| Unusual | 100 years flood | Uplift | 23800.0 | 37485.0 | 1.58 | 1.10 |
| Extreme | | Sliding | 5678.6 | 8258.0 | 1.45 | 1.20 |
| Extreme | | Uplift | 23150.0 | 33360.0 | 1.44 | 1.05 |
| Emergency | | Sliding | 843.4 | 9841.5 | 11.67 | 1.20 |
| Emergency | | Uplift | 28100.0 | 41400.0 | 1.47 | 1.05 |

Remarks:
* N.A. = Not Applicable

| Prepared / Date | Revision |
|---|---|
| Checkeded / Date | Sheet     of |

# Naga Hammadi Barrage Development Consultant

## CALCULATION SHEET

Subject : **Stability Calculation for SLUICEWAY**

Remarks :

* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)
* Load cases according to Design Criteria Memorandum Draft Version : **04.07.99**
* Safety factors according to DIN 1054.
* Weight of water is calculated separetly.

## 1 General

### 1.1 Input Data

| | | | |
|---|---|---|---|
| Angle of internal friction ($\phi°$) | = | 32.5 | |
| Angle bet. horizontal and sliding plane ($\alpha$) | = | 4.0 | |
| Coefficient of friction (Cf) | = | $\tan(\phi+\alpha)$ | |
| | = | 0.740 | |
| Coefficient of earth pres. (k0) | = | $1-\sin(\phi)$ | |
| | = | 0.463 | |

(66.20) or (67.61)

61.83

P1

46.00

42.00

P2

wp1

wp2

21.00     13.00     24.60

58.60

U

Prepared / Date

Revision

Checkeded / Date

Sheet          of

calc. of Sluiceway PoE

18/09/00

# Naga Hammadi Barrage Development Consultant

## CALCULATION SHEET

Subject : **Stability Calculation for SLUICEWAY**
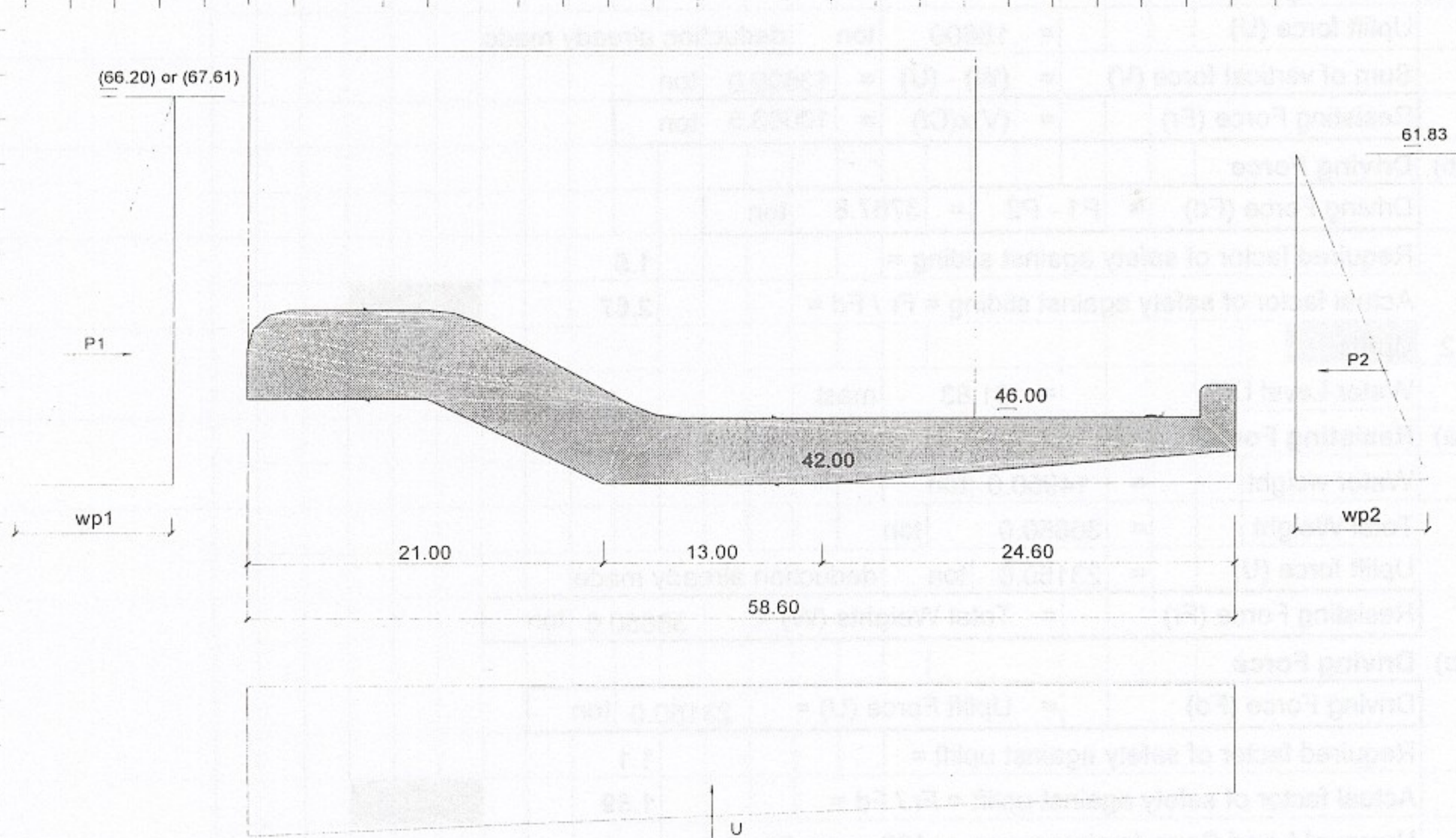
| 1.2 | **Structure Geometry** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Foundation Level | = | 42.00 | | masl | | | | |
| | B | = | 21.00 | m | $q_f$ = | 1.00 | | | |
| | Concrete weight (Wc) | | = | 21900.0 | ton | | | | |

| **2** | **Load Cases** |
|---|---|
| **2.1** | **Normal Load Case** |
| **2.1.1** | **Sliding** |

| | Water Level U/S | | = | 66.20 | masl | Water Level D/S | = | 57.06 | masl |
|---|---|---|---|---|---|---|---|---|---|
| | Water Weight (Ww) | | = | 10300.0 | ton | | | | |
| | Soil Weight (Ws) | | = | 0.0 | ton | | | | |
| | Total Wight (W) | | = | 32200.0 | ton | | | | |

| | wp1 | = | 24.20 | ton/m$^2$ | | | P1 | = | 6149.2 | ton |
|---|---|---|---|---|---|---|---|---|---|---|
| | wp2 | = | 15.06 | ton/m$^2$ | | | P2 | = | 2381.4 | ton |

| a) | **Resisting Force** | | | | | |
|---|---|---|---|---|---|---|
| | Uplift force **(U)** | = | 18600 | ton | deduction already made | |
| | Sum of vertical force **(V)** | = **(W) - (U)** = | 13600.0 | ton | | |
| | Resisting Force (Fr) | = (V)x(Cf) = | 10063.5 | ton | | |

| b) | **Driving Force** | | | | |
|---|---|---|---|---|---|
| | Driving Force (Fd) = P1 - P2 = | 3767.8 | ton | | |
| | Required factor of safety against sliding = | | 1.5 | |
| | Actual factor of safety against sliding = Fr / Fd = | | **2.67** | Safe |

| **2.1.2** | **Uplift** | | | |
|---|---|---|---|---|
| | Water Level D/S | = | 61.83 | masl |

| a) | **Resisting Force** | | | | |
|---|---|---|---|---|---|
| | Water weight | = | 14950.0 | ton | |
| | Total Weight | = | 36850.0 | ton | |
| | Uplift force **(U)** | = | 23150.0 | ton | deduction already made |
| | Resisting Force (Fr) | = Total Weights (W) = | 36850.0 | ton | |

| b) | **Driving Force** | | | |
|---|---|---|---|---|
| | Driving Force (Fd) | = Uplift Force (U) = | 23150.0 | ton |
| | Required factor of safety against uplift = | | 1.1 | |
| | Actual factor of safety against uplift = Fr / Fd = | | **1.59** | Safe |

| **2.2** | **Unusual Load Case** (maintenance or 100 years flood) |
|---|---|
| **2.2.1** | Maintenance |
| **2.2.1.1** | **Sliding** |

| | Water weight | = | 9100.0 | ton | |
|---|---|---|---|---|---|
| | Total Weight | = | 31000.0 | ton | |
| | Uplift force **(U)** | = | 18600 | ton | |
| | Sum of vertical force **(V)** | = **(W) - (U)** = | 12400.0 | ton | |

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| | | | | | |
|---|---|---|---|---|---|
| Resisting Force (Fr) | = | (V) x (Cf) | = | 9175.5 | ton |
| Driving Force (Fd) | = | P1 - P2 | = | 3767.8 | ton |
| Required factor of safety against sliding = | | | | 1.35 | |
| Actual factor of safety against sliding = Fr / Fd = | | | | **2.44** | Safe |

**2.2.1.2 Uplift**

Water Level D/S = 61.83 masl  Water Weight = 12850.0 ton

a) **Resisting Force**

| Resisting Force (Fr) | = | Total Weights (W) = | 34750.0 | ton |
|---|---|---|---|---|

b) **Driving Force**

| Driving Force (Fd) | = | Uplift Force (U) = | 23150.0 | ton |
|---|---|---|---|---|

Required factor of safety against uplift = 1.1

Actual factor of safety against uplift = Fr / Fd = **1.50**  Safe

**2.2.1 100 years flood**

**2.2.2.1 Sliding**

Not applicable

**2.2.2.2 Uplift**

Water Level D/S = 62.50 masl

a) **Resisting Force**

| Water weight | = | 15585.0 | ton | |
|---|---|---|---|---|
| Total Weight | = | 37485.0 | ton | |
| Uplift force **(U)** | = | 23800.0 | ton | deduction already made |
| Resisting Force (Fr) | = | Total Weights (W) = | 37485.0 | ton |

b) **Driving Force**

| Driving Force (Fd) | = | Uplift Force (U) = | 23800.0 | ton |
|---|---|---|---|---|

Required factor of safety against uplift = 1.1

Actual factor of safety against uplift = Fr / Fd = **1.58**  Safe

**2.3 Extreme Case (Maintenance + earthquake)**

**2.3.1 Sliding**

Water Level U/S = 66.20 masl  Water Level D/S = 57.06 masl

$e_h$ = 0.06  H = 24.20

$\gamma$ = 1 ton/m$^3$  B = 21.0 m

Seismic horiz. force due to water pres. ($H_{we}$) = $0.583 \times h^2 \times e^h \times B$

= 430.199 ton  for U/S

= 166.61 ton  for D/S

Seismic factor for vertical load ($e_v$) = 0.04

Seismic hz. force for dead load ($H_{Ge}$) = ($e_h$) x (Wc+Ws) = 1314.0 ton

Seismic vl. force for dead load ($V_{Ge}$) = ($e_v$) x (Wc+Ws+Ww) = 1240.0 ton

a) **Resisting Force**

| Sum of vertical force (Ve) = (V) - ($V_{Ge}$) | = | 11160.0 | ton |
|---|---|---|---|
| Resisting Force (Fr) | = | (Ve) x (Cf) | = | 8258.0 | ton |

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b) **Driving Force**

| Driving Force (Fd) = $H_{ge}+H_{we}+Hwe+(P1-P2)$ | = | 5678.6 | ton |
|---|---|---|---|
| Required factor of safety against sliding = | | 1.20 | |
| Actual factor of safety against sliding = Fr / Fd = | | **1.45** | Safe |

**2.3.2  Uplift**

a) **Resisting Force**

Resisting Force (Fr) = (W) - (VGe) =   33360.0  ton

b) **Driving Force**

| Driving Force (Fd) | = | Uplift force (U) = | 23150.0 | ton |
|---|---|---|---|---|
| Required factor of safety against uplift = | | 1.05 | | |
| Actual factor of safety against uplift = Fr / Fd = | | **1.44** | Safe | |

**2.4      Emergency Releases**

**2.4.1  Sliding**

| Water Level U/S | = | 67.60 | masl | Water Level D/S | = | 65.98 | masl |
|---|---|---|---|---|---|---|---|

a) **Resisting Force**

| Water weight = | | 19000.0 | ton |
|---|---|---|---|
| Total weight | = | 40900.0 | ton |
| Uplift Force (U) | = | 27600.0 | ton |
| Sum of vertical force **(V)** | = | 13300.0 | ton |
| Resisting Force (Fr) | = (V) x (Cf) = | 9841.5 | ton |

b) **Driving Force**

| wp2 | = | 23.98 | ton/m$^2$ | | P1 = | 6037.9 | ton |
|---|---|---|---|---|---|---|---|
| wp1 | = | 25.60 | ton/m$^2$ | | P2 = | 6881.3 | ton |
| Driving Force (Fd) | = | P1 - P2 | = | 843.4 | ton | | |
| Required factor of safety against sliding = | | | | 1.2 | | | |
| Actual factor of safety against sliding = Fr / Fd = | | | | **11.67** | Safe | | |

**2.4.2  Uplift**

a) **Resisting Force**

| Water Level U/S | = | 67.60 | masl | Water Level D/S | = | 66.50 | masl |
|---|---|---|---|---|---|---|---|

a) **Resisting Force**

| Water weight | = | 19500.0 | ton |
|---|---|---|---|
| Total weight | = | 41400.0 | ton |
| Resisting Force (Fr) | = Total Weights (W) = | 41400.0 | ton |

b) **Driving Force**

| Driving Force (Fd) | = | Uplift force (U) = | 28100.0 | ton |
|---|---|---|---|---|
| Required factor of safety against uplift = | | 1.1 | | |
| Actual factor of safety against uplift = Fr / Fd = | | **1.47** | Safe | |

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# Stability Calculation
### for
# POWERHOUSE

## *CALCULATION SHEET*

Final Calculation August 1999

# Naga Hammadi Barrage Development Consultant

# *CALCULATION SHEET*

Subject : **POWERHOUSE** (Summary Sheet)

| Load Case | | Item | Driving Force | Resisting Force | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|
| Normal | | Sliding | 12903.3 | 30856.3 | 2.39 | 1.50 |
| Normal | | Uplift | 61607.8 | 97614.0 | 1.58 | 1.10 |
| Unusual | Maintenance | Sliding | 12903.3 | 23126.3 | 1.79 | 1.35 |
| Unusual | Maintenance | Uplift | 57555.3 | 84997.0 | 1.48 | 1.10 |
| Unusual | 100 years flood | Sliding | N.A. | N.A. | N.A. | 1.35 |
| Unusual | 100 years flood | Uplift | 62177.1 | 97702.0 | 1.57 | 1.10 |
| Extreme | | Sliding | 7269.7 | 15474.2 | 2.13 | 1.20 |
| Extreme | | Uplift | 61607.8 | 81597.1 | 1.32 | 1.05 |
| Emergency | | Sliding | 14276.1 | 36608.4 | 2.56 | 1.20 |
| Emergency | | Uplift | 67951.4 | 105555.0 | 1.55 | 1.05 |

Remarks:
* N.A. = Not Applicable

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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE**

Remarks :

* 1 block of 2 units is considered.
* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)
* weights of any other walls are cosidered.
* Load cases according to Design Criteria Memorandum Draft Version : **04.07.99**
* Safety factors according to DIN 1054.
* Weight of water is calculated separetly.

| 1 | **General** |
|---|---|
| 1.1 | **Input Data** |

Coefficient of friction (Cf)  =  0.637     Coefficient of earth pres. (k0)  =  0.463

# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE**

| 1.2 | **Structure Geometry** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Foundation Level | = | 39.20 | | masl | | | | |
| B | = | 35.40 | m | qf | = | 1.00 | | | |
| | Concrete weight | | = | 75150.0 | | ton | | | |
| | Water inside ph | | = | 18655.0 | | ton | | | |
| | U/S Water weight | | = | 3327.00 | | ton | | | |
| | Total Weight (**W**) | | = | 97132 | | ton | | | |

| **2** | **Load Cases** |
|---|---|
| **2.1** | **Normal Load Case** |
| 2.1.1 | **Sliding** |

| | Water Level U/S | | = | 66.20 | masl | Water Level D/S | = | 57.06 | masl |
|---|---|---|---|---|---|---|---|---|---|

neglect water weight at d/s

| | e1 | = | 27.00 | ton/m$^2$ | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | e2 | = | 17.86 | ton/m$^2$ | | | | | |
| | E1 = | 12903.3 | ton | | | E2 = | 5645.9 | ton | |

**a) Resisting Force**

| | Uplift force (**U**) | | = | 57555.3 | ton | deduction already made | |
|---|---|---|---|---|---|---|---|
| | Sum of vertical force (**V**) | = | (**W**) - (**U**) = | 39576.7 | ton | | |
| | Resisting Force (Fr') | = | (V) x (Cf) + E2 | = | 30856.3 | ton | |

**b) Driving Force**

| | Driving Force (Fd) | = | E1 = | 12903.3 | ton |
|---|---|---|---|---|---|
| | Required factor of safety against sliding = | | | 1.5 | |
| | Actual factor of safety against sliding = Fr / Fd = | | | **2.39** | Safe |

| 2.1.2 | **Uplift** |
|---|---|

| | Water Level D/S | = | 61.83 | masl |
|---|---|---|---|---|

**a) Resisting Force**

| | D/S water weight | = | 482.0 | ton | |
|---|---|---|---|---|---|
| | Total Weight | = | 97614.0 | ton | |
| | Uplift force (**U**) | = | 61607.8 | ton | deduction already made |
| | Resisting Force (Fr) | = | Total Weights (W) = | 97614.0 | ton |

**b) Driving Force**

| | Driving Force (Fd) | = | Uplift Force (U) = | 61607.8 | ton |
|---|---|---|---|---|---|
| | Required factor of safety against uplift = | | | 1.1 | |
| | Actual factor of safety against uplift = Fr / Fd = | | | **1.58** | Safe |

| **2.2** | **Unusual Load Case** (maintenance or 100 years flood) |
|---|---|
| 2.2.1 | Maintenance |
| 2.2.1.1 | **Sliding** |

| | Water inside ph | | = | 6520.0 | ton |
|---|---|---|---|---|---|
| | Total Weight | = | 84997.0 | ton | |
| | Uplift force (**U**) | = | 57555.3 | ton | |
| | Sum of vertical force (**V**) | = | (**W**) - (**U**) = | 27441.7 | ton |

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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Resisting Force (Fr) | | = | (V) x (Cf) + E2 | = | 23126.3 | ton | | |
| Driving Force (Fd) | | = | E1 | = | 12903.3 | ton | | |
| Required factor of safety against sliding = | | | | | 1.35 | | | |
| Actual factor of safety against uplift = Fr / Fd = | | | | | **1.79** | | | |

**2.2.1.2 Uplift**

a) **Resisting Force**

| | | | | | | |
|---|---|---|---|---|---|---|
| Resisting Force (Fr) | | = | Total Weights (W) = | 84997.0 | ton | |

b) **Driving Force**

| | | | | | | |
|---|---|---|---|---|---|---|
| Driving Force (Fd) | | = | Uplift Force (U) = | 57555.3 | ton | |
| Required factor of safety against uplift = | | | | 1.1 | | |
| Actual factor of safety against uplift = Fr / Fd = | | | | **1.48** | | |

**2.2.1 100 years flood**

**2.2.2.1 Sliding**

The water levels for the u/s & D/S are the same as normal load case

**2.2.2.2 Uplift**

| | | | | | |
|---|---|---|---|---|---|
| Water Level D/S | | = | 62.50 | masl | |

a) **Resisting Force**

| | | | | | |
|---|---|---|---|---|---|
| D/S water weight | = | 570.0 | ton | | |
| Total Weight | = | 97702.0 | | ton | |
| Uplift force (U) | = | 62177.1 | ton | deduction already made |
| Resisting Force (Fr) | = | Total Weights (W) = | | 97702.0 | ton |

b) **Driving Force**

| | | | | | |
|---|---|---|---|---|---|
| Driving Force (Fd) | | = | Uplift Force (U) = | 62177.1 | ton |
| Required factor of safety against uplift = | | | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | | | **1.57** | |

**2.3 Extreme Case (Maintenance + earthquake)**

**2.3.1 Sliding**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Water Level U/S | = | 66.20 | masl | | Water Level D/S | = | 57.06 | masl |

| | | | | |
|---|---|---|---|---|
| $e_h$ = | 0.06 | H = | 27.00 | |
| $\gamma$ = 1 ton/m³ | | B = | 35.4 m | |

| | | | |
|---|---|---|---|
| Seismic horiz. force due to water pres. ($H_{we}$) | = | $0.583 \times h^2 \times e^h \times B$ | |
| | = | 902.715 | ton |
| Sesmic factor for horizontal load ($e_h$) | = | 0.06 | |
| Sesmic factor for vertical load ($e_h$) | = | 0.04 | |
| Horiz. Force due to dead load ($H_{Ge}$) = ($e_h$) x (W) = | | 5099.8 | ton |
| Vert. Force due to dead load ($V_{Ge}$) = ($e_v$) x (W) = | | 3399.9 | ton |

a) **Resisting Force**

| | | | | | |
|---|---|---|---|---|---|
| Sum of vertical force (Ve) = (V) - ($V_{Ge}$) = | | | 24041.9 | ton | |
| Resisting Force (Fr) | | = | (Ve) x (Cf) + ($Wp_{ds}$) | = | 15474.2 | ton |

| | |
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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE**

| | b) | **Driving Force** | | | | |
|---|---|---|---|---|---|---|
| | | Driving Force (Fd) = $H_{ge}+2 \times H_{we}+(WP_{us})$ | = | 7269.7 | ton | |
| | | Required factor of safety against sliding = | | 1.20 | | |
| | | Actual factor of safety against sliding = Fr / Fd = | | **2.13** | | Safe |

**2.3.2 Uplift**

a) **Resisting Force**

Resisting Force (Fr) = (W) - (VGe) = 81597.1 ton

b) **Driving Force**

| Driving Force (Fd) | = | Uplift force (U) = | 61607.8 | ton |
|---|---|---|---|---|
| Required factor of safety against uplift = | | | 1.05 | |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.32** | Safe |

## 2.4 Emergency Releases

**2.4.1 Sliding**

| Water Level U/S | = | 67.60 | masl | | Water Level D/S | = | 65.98 | masl |
|---|---|---|---|---|---|---|---|---|

a) **Resisting Force**

| U/S water weight = | 3750.0 | ton | D/s Water weight = | 7500.0 | ton |
|---|---|---|---|---|---|
| Total weight | = | 105055 | ton | | |
| Uplift Force (U) = | 67512.6 | ton | | | |
| Sum of vertical force (**V**) | = | 37542.4 | ton | | |
| e1 = | 26.78 | ton/m$^2$ | E1 = | 12693.9 | ton |
| Resisting Force (Fr) | = | (V) x (Cf) + E1 | = | 36608.4 | ton |

b) **Driving Force**

| e2 = | 28.40 | ton/m$^2$ | E2 = | 14276.11 | ton |
|---|---|---|---|---|---|
| Driving Force (Fd) | = | E2 = | 14276.1 | ton | |
| Required factor of safety against sliding = | | | 1.2 | | |
| Actual factor of safety against sliding = Fr / Fd = | | | **2.56** | Safe | |

**2.4.2 Uplift**

a) **Resisting Force**

| Water Level U/S | = | 67.60 | masl | | Water Level D/S | = | 66.50 | masl |
|---|---|---|---|---|---|---|---|---|

a) **Resisting Force**

| U/S water weight = | 3750.0 | ton | D/s Water weight = | 8000.0 | ton |
|---|---|---|---|---|---|
| Total weight | = | 105555 | ton | | |
| Resisting Force (Fr) | = | Total Weights (W) = | 105555.0 | ton | |

b) **Driving Force**

| Driving Force (Fd) | = | Uplift force (U) = | 67951.4 | ton |
|---|---|---|---|---|
| Required factor of safety against uplift = | | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.55** | Safe |

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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE**

| | | | | | | |
|---|---|---|---|---|---|---|
| Resisting Force (Fr) | = | (V) x (Cf) + E2 | = | 23126.3 | ton |
| Driving Force (Fd) | = | E1 | = | 12903.3 | ton |
| Required factor of safety against sliding = | | | | 1.35 | |
| Actual factor of safety against uplift = Fr / Fd = | | | | **1.79** | |

**2.2.1.2 Uplift**

a) **Resisting Force**

| | | | | |
|---|---|---|---|---|
| Resisting Force (Fr) | = | Total Weights (W) = | 84997.0 | ton |

b) **Driving Force**

| | | | | |
|---|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) = | 57555.3 | ton |
| Required factor of safety against uplift = | | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.48** | |

**2.2.1 100 years flood**

**2.2.2.1 Sliding**

The water levels for the u/s & D/S are the same as normal load case

**2.2.2.2 Uplift**

| | | | |
|---|---|---|---|
| Water Level D/S | = | 62.50 | masl |

a) **Resisting Force**

| | | | | |
|---|---|---|---|---|
| D/S water weight | = | 570.0 | ton | |
| Total Weight | = | 97702.0 | | ton |
| Uplift force (U) | = | 62177.1 | ton | deduction already made |
| Resisting Force (Fr) | = | Total Weights (W) = | 97702.0 | ton |

b) **Driving Force**

| | | | | |
|---|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) = | 62177.1 | ton |
| Required factor of safety against uplift = | | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.57** | |

**2.3 Extreme Case (Maintenance + earthquake)**

**2.3.1 Sliding**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Water Level U/S | = | 66.20 | masl | | Water Level D/S | = 57.06 | masl |
| $e_h$ = | 0.06 | | H = | 27.00 | | | |
| $\gamma$ = 1 ton/m$^3$ | | | B = | 35.4 | m | | |

| | | | |
|---|---|---|---|
| Seismic horiz. force due to water pres. ($H_{we}$) | = | $0.583 \times h^2 \times e^h \times B$ | |
| | = | 902.715 | ton |
| Sesmic factor for horizontal load ($e_h$) | = | 0.06 | |
| Sesmic factor for vertical load ($e_h$) | = | 0.04 | |
| Horiz. Force due to dead load ($H_{Ge}$) = ($e_h$) x (W) = | 5099.8 | ton |
| Vert. Force due to dead load ($V_{Ge}$) = ($e_v$) x (W) = | 3399.9 | ton |

a) **Resisting Force**

| | | | | |
|---|---|---|---|---|
| Sum of vertical force (Ve) = (V) - ($V_{Ge}$) = | | 24041.9 | ton | |
| Resisting Force (Fr) | = | (Ve) x (Cf) + (Wp$_{ds}$) | = | 15474.2 | ton |

| | | |
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# Stability Calculation
### for
# POWERHOUSE ABUTMENT PIER

# *CALCULATION SHEET*

Final Calculation August 1999

# CALCULATION SHEET

Naga Hammadi Barrage Development Consultant

Subject : **POWERHOUSE ABUTMENT PIER** (Summary Sheet)

| Load Case | | Item | Driving Force | | Resisting Force | | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|---|---|
| Normal | | Sliding | 436.7 | | 668.8 | | **1.53** | 1.50 |
| | | Uplift | 467.2 | | 904.8 | | **1.94** | 1.10 |
| | | Overturning | e = | 1.96 | b / 6 = | 2.92 | **safe** | e<b/6 |
| Unusual | Maintenance | Sliding | N.A. | | N.A. | | **N.A.** | 1.35 |
| | | Uplift | N.A. | | N.A. | | **N.A.** | 1.10 |
| | | Overturning | N.A. | | N.A. | | **N.A.** | e<b/6 |
| | 100 years flood | Sliding | N.A. | | N.A. | | **N.A.** | 1.35 |
| | | Uplift | N.A. | | N.A. | | **N.A.** | 1.10 |
| | | Overturning | N.A. | | N.A. | | **N.A.** | e<b/6 |
| Extreme | | Sliding | 530.2 | | 645.7 | | **1.22** | 1.20 |
| | | Uplift | 467.2 | | 868.6 | | **1.86** | 1.05 |
| | | Overturning | e = | 5.57 | b / 3 = | 5.83 | **safe** | e < b/6 |
| Emergency | | Sliding | 436.5 | | 655.6 | | **1.50** | 1.20 |
| | | Uplift | 490.4 | | 904.8 | | **1.84** | 1.05 |
| | | Overturning | e = | 2.05 | b / 3 = | 2.92 | **safe** | e < b/6 |

Remarks:

  * N.A. = Not Applicable

- Dry pier chambers are considered. (no water inside pier)

**CONCLUSION**

- The Abutment Pier in itself is not stable.
- The adjacent PH block will support the pier by applying 2 forces (Fb1), (Fb2) in 2 points at 2 different elevations to reach the above factors of safety.
- The force (**Fb1**) = 220 ton/m at elevation (40.60) masl
- The force (**Fb2**) = 170 ton/m at elevation (57.10) masl

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# Naga Hammadi Barrage Development Consultant

## CALCULATION SHEET

**Subject :** **Stability Calculation for POWERHOUSE ABUTMENT PIER**

Remarks :

* An intermediate part of the pier considered as the critical section.

* 1.0 m long strip is considered.

* The backfill level is taken as the highest level (69.00).

* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)

* Water pressre ($WP_L$) is allowed between the pier and adjacent PH block

* The water pressure ($WP_L$) is calculated and expressed as a percentage ($F_p$) of the water pressure ($WP_R$)

* A sandfill weight factor of 0.75 is considered since not all the chambers are filled.

* Load cases according to Design Criteria Memorandum Draft Version : **04.07.99**

* Safety factors according to DIN 1054.

| 1 | **General** | | | | |
|---|---|---|---|---|---|

### 1.1 Input Data

| | | | |
|---|---|---|---|
| Coefficient of friction ($C_f$) | = | 0.637 | |
| Coefficient of earth pressure ($k_0$) | = | 0.463 | |

### 1.2 Structure Geometry

| | | | |
|---|---|---|---|
| Backfill level (left side) | = | 39.00 | masl |
| Backfill level (right side) | = | 69.00 | masl |
| Foundation Level | = | 38.10 | masl |
| Sandfill level ($\gamma=1.8 t/m3$) | = | 57.00 | masl |
| Concrete top level | = | 69.00 | masl |

$T_s = 1.00$ m , $T_f = 5.00$ m

$T_w = 2.00$ m , $B = 13.50$ m

$q_f = 1.00$ , $q = 2.00$ ton/m$^2$

$F_p = 0.65$



| | | | |
|---|---|---|---|
| Concrete weight | = | 511.50 | ton |
| Cross walls weight | = | 140.00 | ton |
| Sandfill weight | = | 253.33 | ton |
| Water Weight inside pier | = | 0.00 | ton |
| Total Weight (**W**) | = | 904.83 | ton |

| 2 | **Load Cases** |
|---|---|
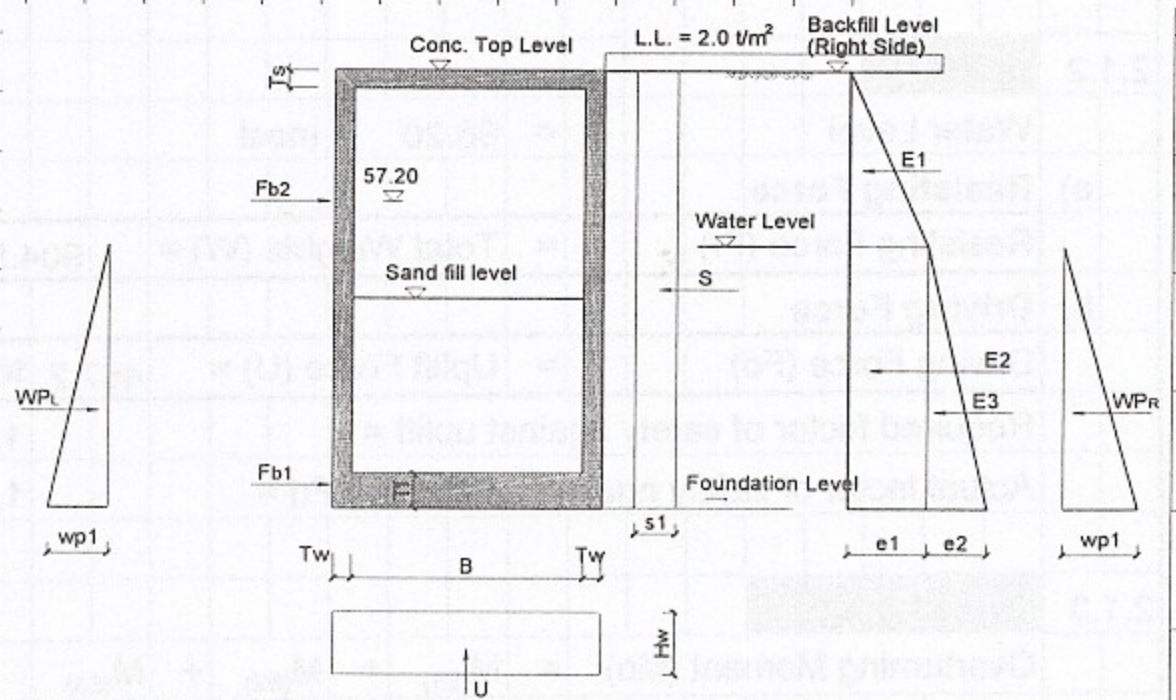
### 2.1 Normal Load Case

#### 2.1.1 Sliding

| | | | | |
|---|---|---|---|---|
| Water Level | | = | 66.20 | masl |
| e1 | = | 2.33 | ton/m$^2$ | |
| e2 | = | 14.31 | ton/m$^2$ | |
| wp1 | = | 28.1 | ton/m$^2$ | |
| s1 | = | 0.93 | ton/m$^2$ | |

| | |
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calc. of powerhouse abutment pier PoE

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Subject : **Stability Calculation for POWERHOUSE ABUTMENT PIER**

| | | | |
|---|---|---|---|
| E1 = | 3.27 | ton | E3 = 201.07 ton |
| E2 = | 65.6 | ton | S = 28.61 ton |
| $WP_R$ = | 394.81 | ton | |
| $WP_L$ = | 256.62 | ton | |

**a) Resisting Force**

| | | | |
|---|---|---|---|
| Uplift force (U) | = | 467.163 | ton |
| Sum of vertical force (V) | = **(W) - (U)** = | 437.67 | ton |
| Resisting Force (Fr') | = (V) x (Cf) = | 278.79 | ton |

assume 2 horizontal forces (Fb1), (Fb2) coming from the powerhouse as shown

assume (Fb2) = 170 ton at distance = 19.0 m from the foundation level

assume (Fb1) = 220 ton at distance = 2.5 m from the foundation level

| | | | |
|---|---|---|---|
| Balance Horiz. force from Powerhouse (Fb) = (Fb1) + (Fb2) | | = | 390 ton |
| Total Resisting Force (Fr) | = (Fr') + (Fb) = | 668.79 | ton |

**b) Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = $E1+E2+E3+S+WP_R-WP_L$ | = | 436.71 ton |
| Required factor of safety against sliding = | | 1.5 | |
| Actual factor of safety against sliding = Fr / Fd = | | **1.53** | Safe |

### 2.1.2 Uplift

Water Level = 66.20 masl

**a) Resisting Force**

| | | | |
|---|---|---|---|
| Resisting Force (Fr) | = Total Weights (W) = | 904.8 | ton |

**b) Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = Uplift Force (U) = | 467.2 | ton |
| Required factor of safety against uplift = | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | **1.94** | Safe |

### 2.1.3 Overturning

| | | | | | | |
|---|---|---|---|---|---|---|
| Overturning Moment (Mo) | = | $M_{(E1)}$ + | $M_{(E2)}$ + | $M_{(E3)}$ + | $M_{(wpr)}$ + | $M_{(s)}$ |
| | = | 94.8 + | 921 + | 1883.4 + | 3698.0 + | 442.1 |

| | | |
|---|---|---|
| Overturning Moment (Mo) | = 7039.6 | ton.m |
| Resisting Moment (Mr) | = $M_{(Fb1)}$ + $M_{(Fb2)}$ + $M_{(wpl)}$ | |
| | 6183.7 | ton.m |
| Resultant Moment (M) | = 855.9 | ton.m |
| Eccentricity (e) = (M) / (V) = | 2.0 | m |
| B / 6 = 2.917 m | > (e) | Safe |

### 2.2 Unusual Load Case (maintenance or 100 years flood)

Remarks :

* No maintenance case is existing.

* The water levels in the 100 years flood case are the same as water levels in normal case.
  So, no calculations required.

| | |
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Subject : **Stability Calculation for POWERHOUSE ABUTMENT PIER**

| 2.3 | **Extreme Load Case (100 years flood + earthquake)** | | |
|---|---|---|---|
| 2.3.1 | Sliding | | |

**Seismic Forces for dead load**

Seismic factor for horizontal load $(e_h)$ =    0.06

Seismic factor for vertical load $(e_v)$ =    0.04

Horizontal force due to dead load $(H_{Ge}) = (e_h) \times (W)$ =    54.29 ton

Vertial force due to dead load $(V_{Ge}) = (e_v) \times (W)$ =    36.19 ton

**Seiemic force for Water Pressure**

$H_{we} = 0.583 \times e_h \times H^2 \times g \times B$

$e_h$ = 0.06     H = 28.10

$\gamma$ = 1 ton/m$^3$     B = 1.0 m

Horizontal force due to water pres. $(H_{we})$    = 27.62 ton

| **Seismic Forces for earth pressure** | = | $0.5(1+k_{v,s})(\gamma \times h + 2q \sin a/(\sin(a+b)) \times h \times k_{a,s}$ |
|---|---|---|

$k_{v,s}$ = 0.04     $k_{a,s}$ = $(k_a + 0.75 k_{h,s})$

$k_{h,s}$ = 0.06     $k_{a,s}$ = 0.508

**Seismic Forces for earth pressure (Es)=**    310.10 ton

| a) | **Resisting Force** |
|---|---|

Sum of vertical force $(Ve) = (V) - (V_{Ge})$ =    401.5 ton

Resisting Force $(Fr)$ = $(Ve) \times (Cf) + (Fb)$ = 645.74 ton

| b) | **Driving Force** |
|---|---|

Driving Force $(Fd) = H_{ge}+H_{we}+Es+WP_R-WP_L$ = 530.19 ton

Required factor of safety against sliding =    1.20

Actual factor of safety against sliding = Fr / Fd =    **1.22**    Safe

| 2.3.2 | Uplift | |
|---|---|---|
| a) | **Resisting Force** | |

Resisting Force $(Fr) = (W) - (VGe)$ =    868.6 ton

| b) | **Driving Force** | |
|---|---|---|

Driving Force $(Fd)$ = Uplift force $(U)$ =    467.2 ton

Required factor of safety against uplift =    1.05

Actual factor of safety against uplift = Fr / Fd =    **1.86**    Safe

| 2.3.3 | Overturning | | | | | | |
|---|---|---|---|---|---|---|---|

Overturning Moment **(Mo)** = $M_{(WPR)}$ + $M_{(HGe)}$ + $M_{(HWe)}$ + $M_{(Es)}$

= 3698.01 + 838.8 + 291.1 + 3593.2

= 8421.1 ton.m

Resisting Moment **(Mr)** = $M_{(fb1+fb2)} + M_{(WPL)}$ = 6183.7 ton.m

Resultanat Moment = $(Mr) - (Mo)$ = 2237.4 ton.m

Eccentricity $(e) = (Mo) / (Ve)$ =    5.6 m

B / 3 = 5.833 m   >   (e)     Safe

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## CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE ABUTMENT PIER**

| 2.4 | **Emergency Releases** |
|---|---|

### 2.4.1 Sliding

| | Water Level | = | 65.98 | masl |
|---|---|---|---|---|

a) **Resisting Force**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $e_1$ | = | 2.52 | ton/m$^2$ | | $wp_1$ | = | 27.9 | ton/m$^2$ |
| $e_2$ | = | 14.20 | ton/m$^2$ | | $s_1$ | = | 0.93 | ton/m$^2$ |

| | | | |
|---|---|---|---|
| $E_1$ = | 3.80 | ton | |
| $E_2$ = | 70.2 | ton | |
| $E_3$ = | 197.94 | ton | |
| $S$ = | 28.61 | ton | |
| $WP_R$ = | 388.65 | ton | |
| $WP_L$ = | 252.62 | ton | |

| | | | |
|---|---|---|---|
| Uplift Force (U) = | | 487.9 | ton |
| Sum of vertical force **(V)** | = | 416.93 | ton |
| Resisting Force (Fr) | = (V) x (Cf) + Fb = | 655.58 | ton |

b) **Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = $E_1+E_2+E_3+S+WP_R-WP_L$ = | 436.55 | ton |
| Required factor of safety against sliding = | | 1.2 | |
| Actual factor of safety against sliding = Fr / Fd = | | **1.50** | Safe |

### 2.4.2 Uplift

a) **Resisting Force**

| | | | |
|---|---|---|---|
| Water Level | = | 67.60 | masl |
| Resisting Force (Fr) | = Total Weights (W) = | 904.8 | ton |

b) **Driving Force**

| | | | |
|---|---|---|---|
| Uplift Force (U) = | 490.438 | ton | |
| Required factor of safety against uplift = | | 1.1 | |
| Actual factor of safety against uplift = Fr / Fd = | | **1.84** | Safe |

### 2.4.3 Overturning

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Overturning Moment (Mo) | = | $M_{(E1+E2+E3+WP+S)}$ | | | | | | |
| | = | 109.8 | + | 978 | + 1839.5 | + | 3611.8 | + 442.1 |
| Overturning Moment (Mo) | = | 6981.4 | ton.m | | | | | |
| Resisting Moment (Mr) | = | 6127.7 | ton.m | | | | | |
| Resultant Moment (**M**) | = | 853.7 | ton.m | | | | | |
| Eccentricity (e) = (M) / (V) = | | 2.05 | m | | | | | |
| B / 6 = | 2.917 | m | > | (e) | | Safe | | |

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# Stability Calculation
### for
# POWERHOUSE U/S ABUTMENT PIER

## *CALCULATION SHEET*

Final Calculation August 1999

# CALCULATION SHEET

Subject : **U/S Abutment Pier** (Summary Sheet)

| Load Case | | Item | Driving Force | | Resisting Force | | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|---|---|
| Normal | | Sliding | 146.7 | | 302.2 | | **2.06** | 1.50 |
| | | Uplift | 493.5 | | 967.9 | | **1.96** | 1.10 |
| | | Overturning | e = | 2.47 | b / 6 = | 2.92 | **Safe** | e < b/6 |
| Unusual | Maintenance | Sliding | N.A. | | N.A. | | **N.A.** | 1.35 |
| | | Uplift | N.A. | | N.A. | | **N.A.** | 1.10 |
| | | Overturning | N.A. | | N.A. | | - | e < b/6 |
| | 100 years flood | Sliding | N.A. | | N.A. | | **N.A.** | 1.35 |
| | | Uplift | N.A. | | N.A. | | **N.A.** | 1.10 |
| | | Overturning | N.A. | | N.A. | | - | e < b/6 |
| Extreme | | Sliding | 336.7 | | 591.9 | | **1.76** | 1.20 |
| | | Uplift | 493.5 | | 929.1 | | **1.88** | 1.05 |
| | | Overturning | e = | 4.82 | b / 3 = | 5.83 | **Safe** | e < b/3 |
| Emergency | | Sliding | 146.7 | | 300.4 | | **2.05** | 1.20 |
| | | Uplift | 518.0 | | 989.6 | | **1.91** | 1.05 |
| | | Overturning | e = | 2.49 | b / 3 = | 5.83 | **Safe** | e < b/3 |

Remarks:
* N.A. = Not Applicable

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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE U/S ABUTMENT PIER**

Remarks :

* The backfill level is taken as the average level.
* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)
* 1.0 m long strip is considered.
* Weights of any other walls were considered.
* For all cases, the D/S water level is not applicable.
* Load cases according to Design Criteria Memorandum Draft Version : **04.07.99**
* Safety factors according to DIN 1054.

## 1 General

### 1.1 Input Data

| | | | |
|---|---|---|---|
| Coefficient of friction (Cf) | = | 0.637 | |
| Coefficient of earth pressure (k0) | = | 0.463 | |

### 1.2 Structure Geometry

| | | | |
|---|---|---|---|
| Backfill level (left side) | = | 41.00 | masl |
| Backfill level (right side) | = | 62.00 | masl |
| Foundation Level | = | 38.00 | masl |
| Sandfill level ($\gamma=1.1t/m3$) | = | 50.00 | masl |
| Concrete top level | = | 69.00 | masl |

$Ts = 1.00\ m$ , $Tf = 2.00\ m$

$Tw = 1.00\ m$ , $B = 15.50\ m$

$qf = 1.00$

| | | | |
|---|---|---|---|
| Concrete weight | = | 271.25 | ton |
| Cross walls weight | = | 120.00 | ton |
| Sandfill weight | = | 170.50 | ton |
| Water weight | = | 406.10 | ton |
| Total Weight (W) | = | 967.85 | ton |



## 2 Load Cases

### 2.1 Normal Load Case

#### 2.1.1 Sliding

| | | | |
|---|---|---|---|
| Water Level | = | 66.20 | masl |

a) **Resisting Force**

| | | | |
|---|---|---|---|
| Uplift Force (U) | = | 493.50 | ton |
| Sum of vertical force (V) | = | 474.35 | ton |
| **Resisting Force (Fr)** | = (V) x (Cf) | = 302.16 | **ton** |

b) **Driving Force**

| | | | |
|---|---|---|---|
| $e1$ = | 12.22 | ton/m$^2$ | |
| $E1$ = | 146.68 | ton | |

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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE U/S ABUTMENT PIER**

| | | | | |
|---|---|---|---|---|
| $p1$ = | 1.53 | ton/m$^2$ | | |
| $P$ = | 2.29 | ton | (to be neglected) | |
| Driving Force (Fd) | | = E1 = | 146.68 | ton |
| Required factor of safety against sliding = | | | | 1.5 |
| Actual factor of safety against sliding = Fr / Fd = | | | | **2.06** |

**2.1.2 Uplift**

| | | | | |
|---|---|---|---|---|
| Water Level | | = | 66.20 | masl |

a) **Resisting Force**

| | | | |
|---|---|---|---|
| Resisting Force (Fr) | = Total Weights (W) = | 967.9 | ton |

b) **Driving Force**

| | | | |
|---|---|---|---|
| Driving Force (Fd) | = Uplift Force (U) = | 493.5 | ton |
| Required factor of safety against uplift = | | | 1.1 |
| Actual factor of safety against uplift = Fr / Fd = | | | **1.96** |

**2.1.3 Overturning**

| | | | |
|---|---|---|---|
| Overturning Moment (Mo) | = M$_{(E1)}$ = | 1173.43 | |

| | | | | |
|---|---|---|---|---|
| Eccentricity (e) = (Mo) / (V) = | | 2.47 | m | |
| B / 6 = | 2.917 | m | > | (e) |

**2.2 Unusual Load Case** (maintenance or 100 years flood)

Remarks :

* No maintenance case is existing.

* The water level in the 100 years flood case is the same as the normal load case, therefore no calculation is needed.

**2.3 Extreme Load Case** (100 years flood + earthquake)

**2.3.1 Sliding**

**Seismic Forces for dead load**

| | | | |
|---|---|---|---|
| Seismic factor for horizontal load (e$_h$) = | 0.06 | | |
| Seismic factor for vertical load (e$_v$) = | 0.04 | | |
| Weight of water inside the pier (Ww) = | 406.10 | ton | |
| Dead Load (W) = | 967.85 | ton | |

| | | |
|---|---|---|
| Horizontal force due to dead load (H$_{Ge}$) = (e$_h$) x (W) = | 58.07 | ton |
| Vertical force due to dead load (V$_{Ge}$) = (e$_v$) x (W) = | 38.71 | ton |

| | | | |
|---|---|---|---|
| **Seismic Forces for earth pressure** | = | $0.5\,(1+k_{v,s})\,(\gamma \times h^2 \times k_{a,s})$ | |
| $k_{v,s}$ = | 0.04 | $k_{a,s}$ = | $(k_a + 0.75\,k_{h,s})$ |
| $k_{h,s}$ = | 0.06 | $k_{a,s}$ = | 0.508 |
| **Seismic Forces for earth pressure (Es)** | = | 167.37 | ton |

| | | |
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# CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE U/S ABUTMENT PIER**

**Seismic force for Water Pressure**

$H_{we} = 0.583 \times e_h \times H^2 \times g \times B$

| | | | | |
|---|---|---|---|---|
| $e_h$ = | 0.06 | | $H$ = | 28.20 |
| $\gamma = 1$ ton/m$^3$ | | | $B$ = | 1.0 m |

Horizontal force due to water pres. ($H_{we}$) = 27.82 ton

a) **Resisting Force**

Sum of vertical force ($Ve$) = ($Wt$) - ($V_{Ge}$) = 929.1 ton

Resisting Force ($Fr$) = ($Ve$) x ($Cf$) = 591.86 ton

b) **Driving Force**

Driving Force ($Fd$) = $H_{ge} + 4 \times H_{we} + Es$ = 336.71 ton

Required factor of safety against sliding = 1.20

Actual factor of safety against sliding = $Fr / Fd$ = **1.76**

2.3.2 **Uplift**

a) **Resisting Force**

Resisting Force = ($Ve$) = 929.1 ton

b) **Driving Force**

Driving Force ($Fd$) = Uplift Force ($U$) = 493.5 ton

Required factor of safety against uplift = 1.05

Actual factor of safety against uplift = $Fr / Fd$ = **1.88**

2.3.3 **Overturning**

Overturning Moment (**Mo**) = $M_{(HGe)} + M_{(HWe)} + M_{(Es)}$

= 900.1 + 1569 + 2008

Overturning Moment (**Mo**) = 4477.5 ton.m

Eccentricity ($e$) = ($Mo$) / ($Ve$) = 4.82 m

$B / 3$ = 5.833 m > ($e$)

2.4 **Emergency Releases**

2.4.1 **Sliding**

Water Level = 67.60 masl

a) **Resisting Force**

Weight of water inside the pier ($Ww$) = 427.80 ton

Uplift force ($U$) = 518.0 ton

Sum of vertical force (**V**) = 471.55 ton

Resisting Force ($Fr$) = ($V$) x ($Cf$) = 300.38 ton

b) **Driving Force**

$e1$ = 12.22 ton/m$^2$

$E1$ = 146.68 ton

Driving Force ($Fd$) = $E1$ = 146.68 ton

Required factor of safety against sliding = 1.2

Actual factor of safety against sliding = $Fr / Fd$ = **2.05**

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| 2.4.2 | Uplift | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | a) | **Resisting Force** | | | | | | | | | |
| | | Water Level | | | = | 67.60 | | masl | | | |
| | | | Resisting Force (Fr) | | = | Total Weights (W) = | | | 989.6 | ton | |
| | b) | **Driving Force** | | | | | | | | | |
| | | | Driving Force (Fd) | | = | Uplift Force (U) | = | 518.00 | ton | | |
| | | | | | | | | | | | |
| | | Required factor of safety against uplift = | | | | | | | 1.1 | | |
| | | Actual factor of safety against uplift = Fr / Fd = | | | | | | | **1.91** | | |

| 2.4.3 | Overturning | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Overturning Moment (**Mo**) = | | 1173.4 | ton.m | | | | |
| | Eccentricity (e) = (Mo) / (v) = | | 2.49 | m | | | | |
| | B / 3 = | 5.833 | m | > | (e) | | Safe | |

Subject : **Stability Calculation for POWERHOUSE U/S ABUTMENT PIER**

**2.4.2** Uplift

  a) **Resisting Force**

    Water Level $=$ 67.60 masl

      Resisting Force (Fr) $=$ Total Weights (W) $=$ 989.6 ton

  b) **Driving Force**

    Driving Force (Fd) $=$ Uplift Force (U) $=$ 518.00 ton

    Required factor of safety against uplift =    1.1

    Actual factor of safety against uplift = Fr / Fd =    **1.91**

**2.4.3** Overturning

  Overturning Moment (Mo) =   1173.4 ton.m

  Eccentricity (e) = (Mo) / (v) =   2.49 m

  B / 3 =   5.833 m   >   (e)      Safe

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# Stability Calculation
### for
# POWERHOUSE D/S ABUTMENT PIER

## *CALCULATION SHEET*

### Final Calculation August 1999

# CALCULATION SHEET

Subject : **D/S Abutment Pier** (Summary Sheet)

| Load Case | | Item | Driving Force | Resisting Force | Actual Factor of Safety | Required Factor of Safety |
|---|---|---|---|---|---|---|
| Normal | | Sliding | 170.7 | 307.7 | **1.80** | 1.50 |
| | | Uplift | 358.4 | 831.8 | **2.32** | 1.10 |
| | | Overturning | e = 2.79 | b / 6 = 2.92 | **Safe** | e < b/6 |
| Unusual | Maintenance | Sliding | N.A. | N.A. | **N.A.** | 1.35 |
| | | Uplift | N.A. | N.A. | **N.A.** | 1.10 |
| | | Overturning | N.A. | N.A. | - | e < b/6 |
| | 100 years flood | Sliding | N.A. | N.A. | **N.A.** | 1.35 |
| | | Uplift | 327.8 | 842.2 | **2.57** | 1.10 |
| | | Overturning | N.A. | N.A. | - | e < b/6 |
| Extreme | | Sliding | 221.3 | 515.0 | **2.33** | 1.20 |
| | | Uplift | 370.1 | 808.5 | **2.18** | 1.05 |
| | | Overturning | e = 3.14 | b / 3 = 5.83 | **Safe** | e < b/3 |
| Emergency | | Sliding | 119.4 | 296.3 | **2.48** | 1.20 |
| | | Uplift | 112.3 | 576.4 | **5.13** | 1.05 |
| | | Overturning | e = 1.49 | b / 3 = 5.83 | **Safe** | e < b/3 |

Remarks:
* N.A. = Not Applicable

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summary of Powerhouse ds Pier Poe

18/09/00

## Naga Hammadi Barrage Development Consultant

### CALCULATION SHEET

**Subject : Stability Calculation for POWERHOUSE D/S ABUTMENT PIER**

Remarks :

* The first 16.0 m of the pier considered as the critical section.
* The backfill level is taken as the average level.
* A concrete quantity factor ($q_f$) is considered to allow for load verification (in this case = 1.0)
* 1.0 m long strip is considered.
* Weights of any other walls were cosidered.
* For all cases, the U/S water level is not applicable.
* Load cases according to Design Criteria Memorandum Draft Version : **04.07.99**
* Safety factors according to DIN 1054.

| 1 | General | | | | | | |
|---|---|---|---|---|---|---|---|
| 1.1 | **Input Data** | | | | | | |
| | Coefficient of friction (Cf) | | | = | 0.637 | | |
| | Coefficient of earth pressure (k0) | | | = | 0.463 | | |
| 1.2 | **Structure Geometry** | | | | | | |
| | Backfill level (left side) | | = | 45.55 | masl | | |
| | Backfill level (right side) | | = | 63.00 | masl | | |
| | Foundation Level | | = | 41.35 | masl | | |
| | Sandfill level ($\gamma$=1.1t/m3) | | = | 50.30 | masl | | |
| | Concrete top level | | = | 69.00 | masl | | |
| | Ts = | 1.00 | m , | Tf = | 4.00 | m | |
| | Tw = | 1.00 | m , | B = | 15.50 | m | |
| | qf = | 1.00 | | | | | |
| | Concrete weight | | = | 332.00 | ton | | |
| | Cross walls weight | | = | 160.00 | ton | | |
| | Sandfill weight | | = | 84.40 | ton | | |
| | Total Weight (W) | | = | 576.40 | ton | | |



| 2 | **Load Cases** | | | | | |
|---|---|---|---|---|---|---|
| 2.1 | **Normal Load Case** | | | | | |
| 2.1.1 | **Sliding** | | | | | |
| | Water Level | | = | 57.06 | masl | |
| a) | **Resisting Force** | | | | | |
| | Uplift Force (U) | = | 274.93 | ton | | |
| | Water Weight inside pier = | | 181.51 | ton | | |
| | Sum of vertical force (V) | = | 482.98 | ton | | |
| | **Resisting Force (Fr)** | = | (V) x (Cf) | = | 307.66 | ton |
| b) | **Driving Force** | | | | | |
| | e1 = | 5.78 | ton/m$^2$ | e2 = | 8.00 | ton/m$^2$ |
| | E1 = | 17.15 | ton | | | |
| | E2 = | 90.73 | ton | | | |
| | E3 = | 62.85 | ton | | | |

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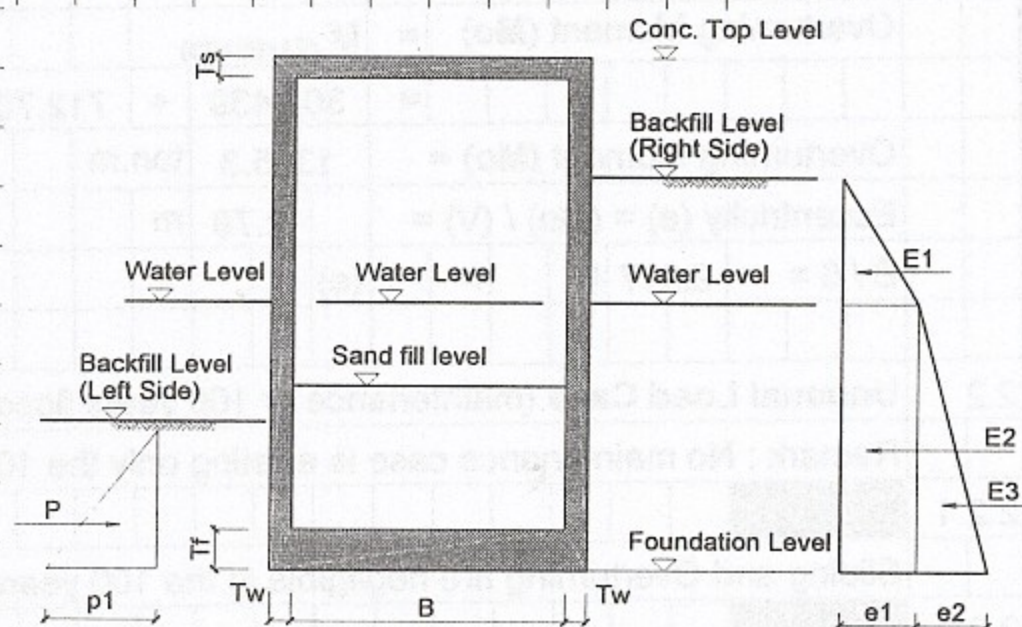
calc. of Powerhouse ds Pier Poe

18/09/00

Subject : **Stability Calculation for POWERHOUSE D/S ABUTMENT PIER**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| p1 = | 2.14 | ton/m$^2$ | | | P = | 4.49 | ton | (to be neglected) | | |
| Driving Force (Fd) | | = | E1+E2+E3 | = | 170.73 | ton | | | |
| Required factor of safety against sliding = | | | | 1.5 | | | |
| Actual factor of safety against sliding = Fr / Fd = | | | | **1.80** | | | |

**2.1.2 Uplift**

| | | | | | |
|---|---|---|---|---|---|
| Water Level | = | 61.83 | masl | | |
| Water Weight inside pier = | 255.44 | ton | | | |

a) **Resisting Force**

| | | | | | |
|---|---|---|---|---|---|
| Resisting Force (Fr) | = | Total Weights (W) = | 831.8 | ton | |

b) **Driving Force**

| | | | | | |
|---|---|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) | = | 358.4 | ton |
| Required factor of safety against uplift = | | | 1.1 | | |
| Actual factor of safety against uplift = Fr / Fd = | | | **2.32** | Safe | |

**2.1.3 Overturning**

| | | | | | |
|---|---|---|---|---|---|
| Overturning Moment (Mo) | = | M $_{(E1+E2+E3)}$ | | | |
| | = | 303.439 | + 712.704 | + 329.12 | |
| Overturning Moment (Mo) = | | 1345.3 | ton.m | | |
| Eccentricity (e) = (Mo) / (V) = | | 2.79 | m | | |
| B / 6 = | 2.917 | m | > | (e) | Safe |

**2.2 Unusual Load Case** (maintenance or 100 years flood)

Remark : No maintenance case is existing only the 100 years flood case is considered

**2.2.1 Sliding**

Sliding and Overturning are negligible in the 100 years flood case.

**2.2.2 Uplift**

a) **Resisting Force**

| | | | | | |
|---|---|---|---|---|---|
| Water Level | = | 62.50 | masl | | |
| Resisting Force (Fr) | = | Total Weights (W) = | 842.2 | ton | |

b) **Driving Force**

| | | | | | |
|---|---|---|---|---|---|
| Driving Force (Fd) | = | Uplift Force (U) | = | | 327.8 | ton |
| Required factor of safety against uplift = | | | 1.1 | | |
| Actual factor of safety against uplift = Fr / Fd = | | | **2.57** | Safe | |

**2.3 Extreme Load Case** (100 years flood + earthquake)

**2.3.1 Sliding**

**Seismic Forces for dead load**

| | | | |
|---|---|---|---|
| Seismic factor for horizontal load (e$_h$) = | | 0.06 | |
| Seismic factor for vertical load (e$_v$) = | | 0.04 | |
| Weight of water inside the pier (Ww) = | | 265.825 | ton |
| Total weight = | 842.22 | ton | |
| Horizontal force due to dead load (H$_{Ge}$) = (e$_h$) x (Wt) = | | 50.53 | ton |
| Vertial force due to dead load (V$_{Ge}$) = (e$_v$) x (Wt) = | | 33.69 | ton |

| | |
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| Prepared / Date | Revision |
| Checked / Date | Sheet of |

## CALCULATION SHEET

Subject : **Stability Calculation for POWERHOUSE <u>D/S ABUTMENT</u> PIER**

| | | |
|---|---|---|
| **Seismic Forces for earth pressure** | = | $0.5\,(1+k_{v,s})\,(\gamma \times h^2 \times k_{a,s})$ |

| | | | | | |
|---|---|---|---|---|---|
| $k_{v,s}$ | = | 0.04 | $k_{a,s}$ | = | $(k_a + 0.75\,k_{h,s})$ |
| $k_{h,s}$ | = | 0.06 | $k_{a,s}$ | = | 0.508 |

| Seismic Forces for earth pressure (Es) = | 136.2 | ton |
|---|---|---|

**Seismic force for Water Pressure**

$H_{we} = 0.583 \times e_h \times H^2 \times g \times B$

| | | | | | |
|---|---|---|---|---|---|
| $e_h$ = | 0.06 | | H = | 15.71 | |
| $\gamma = 1\ \text{ton/m}^3$ | | | = | 1.0 | m |

| Horizontal force due to water pres. ($H_{we}$) | = | 8.63 | ton |
|---|---|---|---|

a) **Resisting Force**

| Sum of vertical force (Ve) = (Wt) - ($V_{Ge}$) = | 808.5 | ton |
|---|---|---|

| Resisting Force (Fr) | = | (Ve) x (Cf) | = | 515.04 | ton |
|---|---|---|---|---|---|

b) **Driving Force**

| Driving Force (Fd) = $H_{ge}$ + 4x$H_{we}$ + Es = | 221.27 | ton |
|---|---|---|

| Required factor of safety against sliding = | 1.20 |
|---|---|
| Actual factor of safety against sliding = Fr / Fd = | **2.33** | Safe |

**2.3.2  Uplift**

a) **Resisting Force**

| Resisting Force = (Ve) = | 808.5 | ton |
|---|---|---|

b) **Driving Force**

| Driving Force (Fd) | = | Uplift Force (U) | = | 370.13 | ton |
|---|---|---|---|---|---|

| Required factor of safety against uplift = | 1.05 |
|---|---|
| Actual factor of safety against uplift = Fr / Fd = | **2.18** | Safe |

**2.3.3  Overturning**

| Overturning Moment (Mo) | = | $M_{(HGe)}$ | + | 4x$M_{(HWe)}$ | + | $M_{(Es)}$ |
|---|---|---|---|---|---|---|
| | = | 698.6 | + | 365.2 | + | 1474.4 |

| Overturning Moment (Mo) = | 2538.2 | ton.m |
|---|---|---|

| Eccentricity (e) = (Mo) / (W) = | 3.14 | m |
|---|---|---|

| B / 3 = | 5.833 | m | > | (e) | | Safe |
|---|---|---|---|---|---|---|

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calc. of Powerhouse ds Pier Poe                                                                 18/09/00

## CALCULATION SHEET

**Subject : Stability Calculation for POWERHOUSE D/S ABUTMENT PIER**

| 2.4 | **Emergency Releases** | | | | | |
|---|---|---|---|---|---|---|
| 2.4.1 | Sliding | | | | | |
| | Water Level | = | 65.98 | masl | | |
| a) | **Resisting Force** | | | | | |
| | Reduction of weight due to submerged concrete part | = | 111.26 | ton | | |
| | Sum of vertical force (V) | = | 465.14 | ton | | |
| | Resisting Force (Fr) | = (V) x (Cf) = | 296.29 | ton | | |
| b) | **Driving Force** | | | | | |

$e1 = 11.03 \ \text{ton/m}^2$

$E1 = 119.36 \ \text{ton}$

Driving Force (Fd) $= E1 = 119.36$ ton

Required factor of safety against sliding = 1.2

Actual factor of safety against sliding = Fr / Fd = **2.48**  Safe
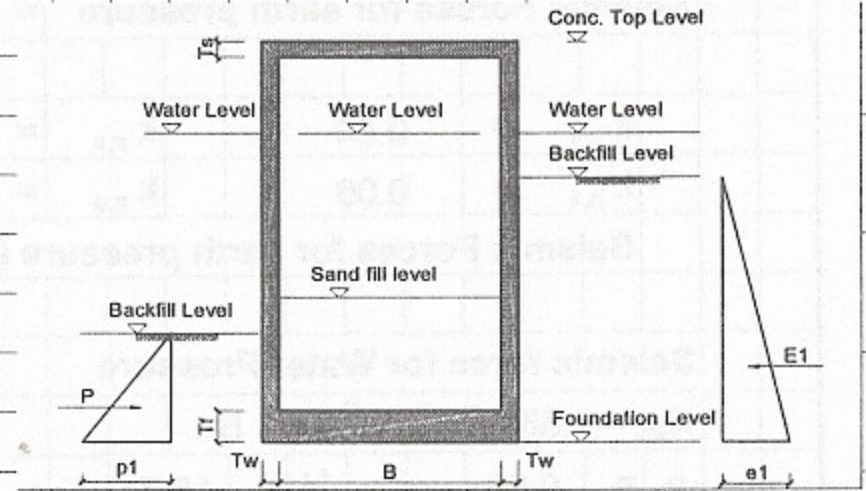
| 2.4.2 | Uplift | | | | |
|---|---|---|---|---|---|
| a) | **Resisting Force** | | | | |
| | Water Level | = | 66.50 | masl | |
| | Resisting Force (Fr) | = | Total Weights (W) = | 576.4 | ton |
| b) | **Driving Force** | | | | |
| | Driving Force (Fd) | = | Vol. of submerged concrete part = | 112.3 | ton |

Required factor of safety against uplift = 1.1

Actual factor of safety against uplift = Fr / Fd = **5.13**  Safe

| 2.4.3 | Overturning | | | |
|---|---|---|---|---|
| | Overturning Moment (Mo) = | 861.4 | ton.m | |
| | Eccentricity (e) = (Mo) / (W) = | 1.49 | m | |
| | B / 3 = | 5.833 | m | > | (e) |

Safe

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|---|---|---|---|
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# Appendix 6.2

# Typical Arrangement of Reinforcement

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## DRAWINGS:

A 6.2 - 1    Navigation Lock

A 6.2 - 2    Sluiceway

A 6.2 - 3    Public Road Bridge

A 6.2 - 4    Powerhouse

NO 1

NO 2

Ø20 /15cm
Ø20 /15cm

Ø20 /15cm                    Ø20 /15cm                                                                    Ø20 /15cm                    Ø20 /15cm

2L Ø22 /15cm          2L Ø22 /15cm                                                                        Ø20 /50cm                    2L Ø20 /50cm

Ø20 /50cm             Ø20 /50cm                    Ø20 /15cm        Ø20 /15cm                    Ø20 /50cm             Ø20 /50cm             3L Ø25 /15cm

Ø20 /15cm
Ø20 /15cm

Ø20 /50cm                                                Ø20 /15cm                          Ø20 /50cm

Ø20 /15cm        3L Ø25 /15cm                                                            3L Ø25 /15cm        Ø20 /15cm

Ø20 /50cm                                          Ø25                              Ø20 /50cm

Ø28

Ø20 /50cm        3L Ø25 /15cm        Ø22                3L Ø25 /15cm        Ø20 /50cm

Ø20 /15cm                                                        Ø20 /15cm

3.00

1.50
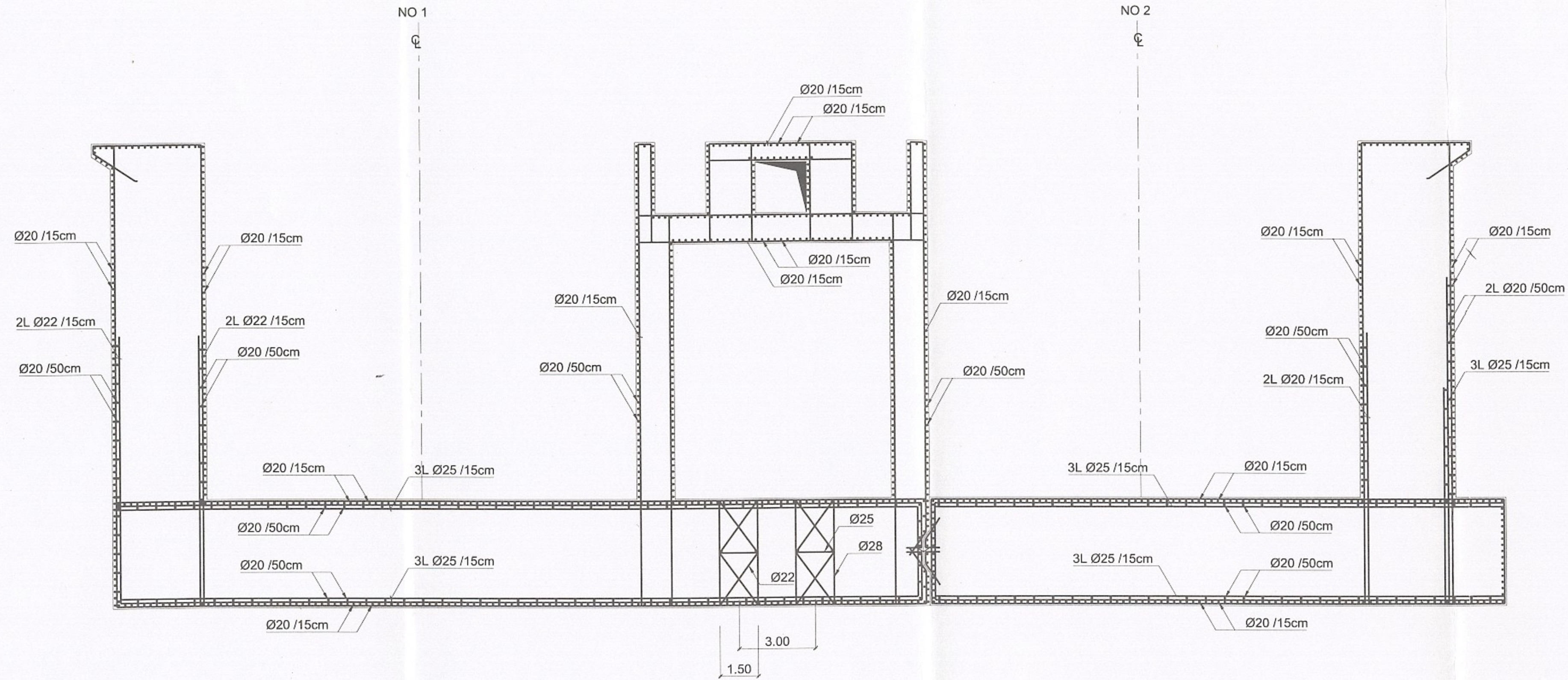
**NAVIGATION LOCK SECTION**

SCALE

0    2    4    6    8    10    12 m

(ORIG. SCALE : 1/200)

**SLUICEWAY FOUNDATION SALB**

Labels on foundation slab drawing:
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 24 Ø22
- 4 Ø22
- 24 Ø22
- 6 Ø22
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- Ø25
- Ø28
- Ø22
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- 1.50
- 3.00
- ₵ OF ROAD
- ₵ OF PROJECT
- 46.00
- 48.00
- 7 Ø22
- Ø22 /15cm
- Ø22 /15cm
- Ø22 /15cm
- 2L Ø22 /15cm
- Ø22 /15cm
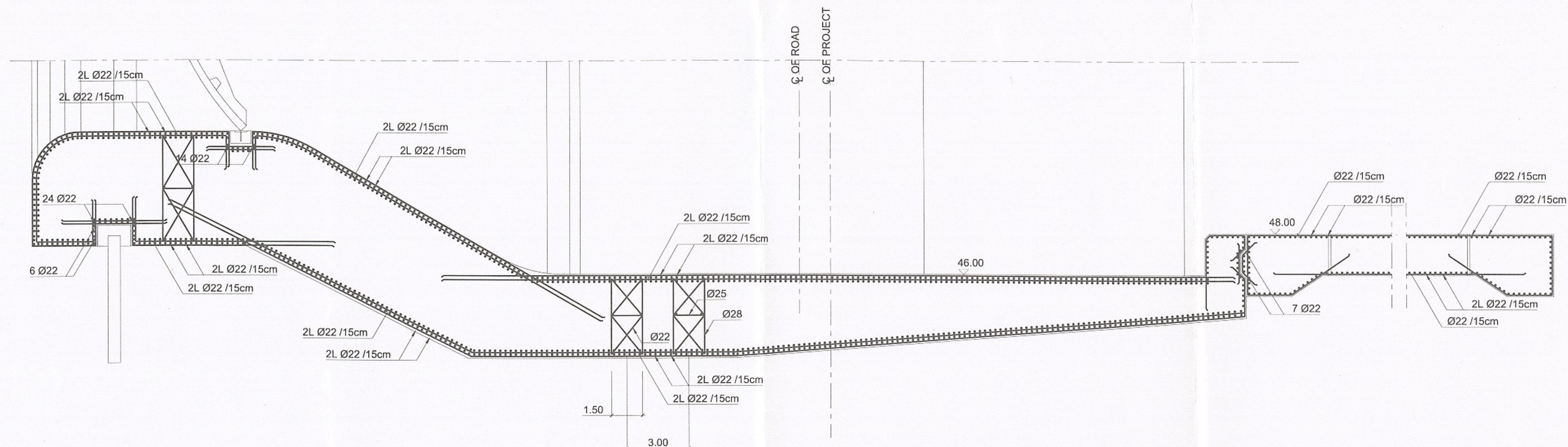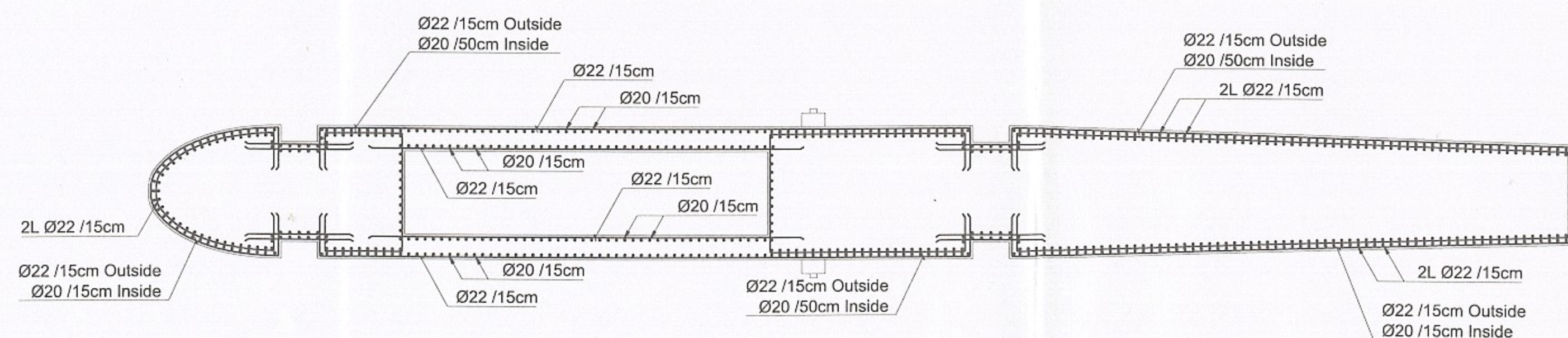
**SLUICEWAY PIER**

**ELEV. 65.50**

Labels on pier drawing:
- Ø22 /15cm Outside / Ø20 /50cm Inside
- Ø22 /15cm
- Ø20 /15cm
- Ø22 /15cm Outside / Ø20 /50cm Inside
- 2L Ø22 /15cm
- 2L Ø22 /15cm
- Ø20 /15cm
- Ø22 /15cm
- Ø22 /15cm
- Ø20 /15cm
- Ø22 /15cm Outside / Ø20 /15cm Inside
- Ø20 /15cm
- Ø22 /15cm
- Ø22 /15cm Outside / Ø20 /50cm Inside
- 2L Ø22 /15cm
- Ø22 /15cm Outside / Ø20 /15cm Inside

**SCALE**

0  2  4  6  8  10  12 m

(ORIG. SCALE : 1/200)

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SLUICEWAY

SA/PZ/FEB. 2000

APPR.:          DATE

TYPICAL REINFORCEMENT

DRAWING NO          540014

S 04-01

Ø16 /15cm    11 Ø25    Ø16 /15cm
Ø12 /15cm    Ø12 /15cm
Ø16 /15cm
₡ BRIDGE

Ø12 /15cm
2 Ø16    Ø10 /15cm
Ø16 /15cm
2 Ø16
Ø12 /15cm
2 Ø16    Ø16 /15cm

3L x11Ø32

**TYPICAL SECTION FOR SPAN 21.0 m**



Ø12 /15cm
Ø16 /15cm
4 Ø20

2 Ø12
2 Ø12    Ø10 /15cm    Ø16 /15cm
2 Ø12    Ø12 /15cm
2 Ø12

7 Ø25

**TYPICAL SECTION FOR EDGE BEAM**



Ø16 /15cm    9 Ø25    Ø16 /15cm
Ø12 /15cm    Ø12 /15cm
₡ BRIDGE

Ø12 /15cm
2 Ø16    Ø10 /15cm
Ø16 /15cm
2 Ø16    Ø12 /15cm
2 Ø16    Ø16 /15cm

3L x9Ø32
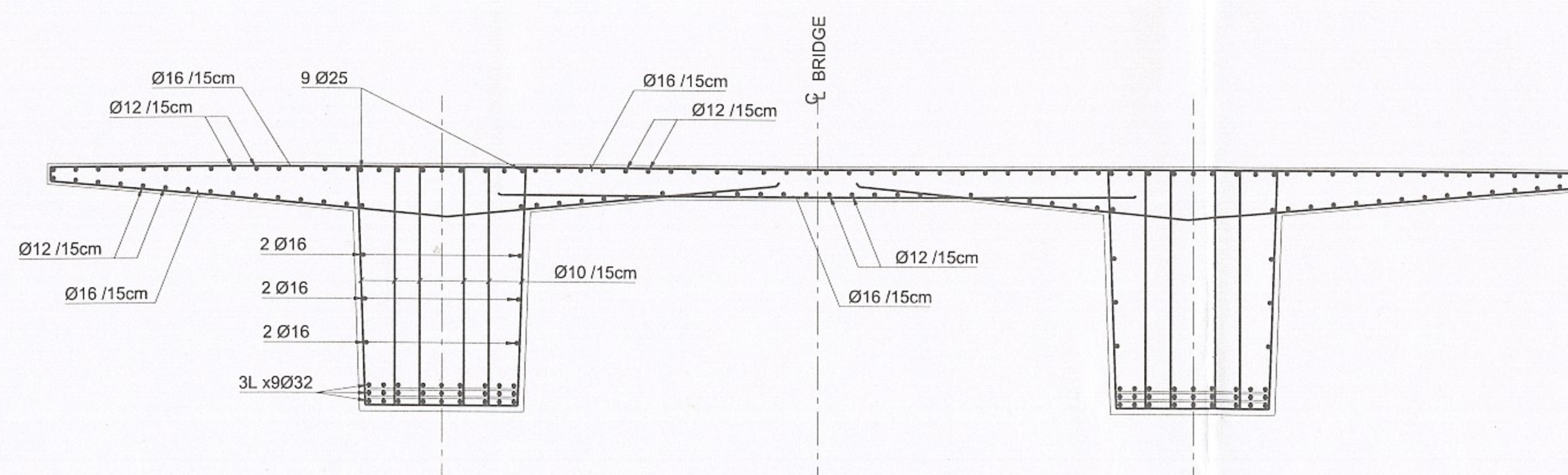
**TYPICAL SECTION FOR SPAN 19.0 m**

SCALE

0   0.5   1.0   1.5   2.0   2.5   3.0 m

(ORIG. SCALE : 1/50)
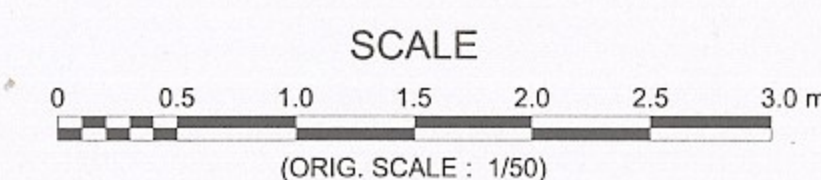
**New Naga Hammadi Barrage
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MPWWR - MEE

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SA/PZ/FEB. 2000

APPR:    DATE

**PUBLIC ROAD BRIDGE**

TYPICAL REINFORCEMENT

DRAWING NO   340014

**S 06-01**

Mesh Ø16 /1.0x1.0x1.0 m

Ø22 /15cm
Ø22 /15cm

Ø16 /15cm
Ø16 /15cm

Ø16 /15cm
Ø16 /15cm

Ø16 /15cm

Ø16 /15cm
Ø20 /15cm
Ø20 /15cm
Ø25 /15cm
Ø12 /20cm
4 Ø20

Ø20 /15cm

2 Ø16
2 Ø16
Ø16 /20cm
2 Ø16

Ø20 /15cm
Ø20 /15cm

7 Ø20

Ø20 /15cm

Ø16 /15cm
Ø16 /15cm
Ø25 /15cm

Ø22 /15cm
2L Ø25 /15cm

Ø16 /15cm
Ø16 /15cm

Ø22 /15cm
Ø22 /15cm

Ø12 /15cm
Ø16 /15cm

Ø22 /15cm
2L Ø20 /15cm
2L Ø20 /15cm

Ø16 /15cm
Ø16 /15cm

2L Ø22 /15cm

2L Ø20 /15cm
Ø22 /15cm

2L Ø20 /15cm
2L Ø20 /15cm
Ø22 /15cm

Ø22 /15cm
Ø22 /15cm

Ø22 /15cm
2L Ø22 /15cm

Ø22 /15cm
Ø22 /15cm

Ø22 /15cm

Ø22 /15cm
Ø28 /15cm
2L Ø28 /15cm

Ø22 /15cm
Ø28 /15cm

Ø28 /15cm
2L Ø28 /15cm

2L Ø28 /15cm
2L Ø28 /15cm

2L Ø28 /15cm
2L Ø28 /15cm

2L Ø28 /15cm

Ø25
Ø28

2L Ø28 /15cm
2L Ø28 /15cm

Ø25
Ø28
Ø22

Ø22
Ø28
Ø25

3.00
1.50

2L Ø28 /15cm
2L Ø28 /15cm

2L Ø28 /15cm

2L Ø28 /15cm
2L Ø28 /15cm

2L Ø28 /15cm
2L Ø28 /15cm

3.00
1.50

**POWERHOUSE SECTION**

SCALE

0  2  4  6  8  10  12 m

(ORIG. SCALE : 1/200)

**New Naga Hammadi Barrage and Hydropower Plant**
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**POWERHOUSE**
TYPICAL REINFORCEMENT

SA/PZ/Feb. 2000

APPR.:   DATE

DRAWING NO.   340914

**S 03-01**

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# CHAPTER 7

# APPROACH TO DESIGN AND STABILITY OF GEOTECHNICAL WORKS

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Appendix 7.2: Seepage Analyses under and around Permanent Structures

Appendix 7.3: Construction Pit – Safety against Uplift

## LIST OF FIGURES

## LIST OF TABLES

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# 7. APPROACH TO DESIGN AND STABILITY OF GEOTECHNICAL WORKS

## 7.1 General Geotechnical Features

Construction of the New Naga Hammadi Barrage and Hydropower Plant requires the establishment of a deep and dry construction pit in the River Nile. The barrage with sluiceway, hydropower plant and double navigation lock of total length of 340 m is spanning over the entire river cross section between both riverbanks. The width of the river is some 250 m, with the riverbed level ranging between 51 and 52 m asl. The water level seasonally varies between 58 and 62 m asl.

The powerhouse with a width of 70.8 m and a length of 71 m together with the adjoining piers will be found at 38.0 m asl. The deepest foundation level of the 88.6 m long sluiceway, is at 42.0 m asl. The navigation lock chambers are found at 50.6 m asl, and the filling system adjacent to the sluiceway is found at 43.6 m asl like the sluiceway abutment pier.

A cut-off wall under all concrete structures will be constructed with a depth of between 25 and 30 m for the intersection of seepage. The cut-off wall is laterally extended to intersect seepage around the barrage.

For the deepest foundation, a drawdown of some 24 m below the maximum river water level is required, and must be maintained for about two years. The total bottom area of the dry construction pit is some 115,000 m$^2$.

Sealed Cofferdams have to be constructed in the river under water in a depth between some 7 to 10 m to route the river through a diversion canal around the dry construction pit. A diaphragm wall around the entire pit has to be constructed along the river banks and through the cofferdams. After removal of the cofferdams, the river will flow through the sluiceway so that the upstream part of the diversion canal can be backfilled after placing of the closure dam. Backfill has to be placed mainly under water in a depth between 6 and 9 m. Deep vibro-compaction shall be applied to this backfill in order to achieve sufficient stability of the slopes and provide for construction of the administration compound on this backfill.

## 7.2 Embankments and Backfill

### 7.2.1 Right Bank Closure Dyke

The right bank closure dyke cuts off the flood channel and ensures dry access to the construction site from the national highway right from the beginning of construction. The dyke will consist of silty material from excavations on the right bank with a sand and gravel zone on the slope facing the river. The water level within this layer will follow changes of the river water level and will also act as a filter between the dyke and the slope protection.

Slope protection will be placed up to 68 m asl, providing the necessary freeboard for wind set-up and wave run-up even during the emergency release of 7,000 m$^3$/s.

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During construction, the dyke is not subject to a significant hydraulic gradient, since the floodway or the fill in the floodway is impounded from downstream. After impoundment, this dyke will perform like a surface sealing of the mainly permeable floodway fill. The foundation level of the right bank closure dyke will be at a level of 60 m asl or below. Aquatic vegetation has to be removed and the foundation has to be graded and compacted for subsequent dyke foundation during the low flow period in December and January.

Excavation material from the Holocene silt and clay will be suitable for dyke construction. The material has to be placed, graded and compacted in layers not exceeding 25 cm. The compliance of material properties with the technical specifications has to be checked by soil mechanic laboratory testing.

The results of the slope stability calculations are included in **Appendix 7-1**.

### 7.2.2 Temporary Cofferdams

Temporary cofferdams upstream and downstream of the construction pit are required to establish a deep and dry pit and route the river through the diversion canal. The cofferdams are temporary works and have to be designed by the Contractor. In general, rockfill bunds should be placed first in order to initially stabilize a cofferdam in the river. Placing of material can be performed by end-dumping from one or both riverbanks by trucks as well as by dumping of rockfill through barges with hydraulic flaps. Before the final closure can be performed by only end-dumping, a sill of sufficient height and stability must have been established as river bed protection.

Prior to river closure by the upstream cofferdam it would be desirable to establish a rockfill sill in the river at the location of the downstream cofferdam, because such a sill would ease the routing of the river through the diversion canal and reduce the high velocities during final closure.

From a dry working platform of sufficient elevation, the rockfill bunds will be extended towards upstream by sand and gravel fill. This fill is required for both, achieving the final height of the cofferdams and enabling trenching for cut-off wall construction. Trenching a cut-off wall through rockfill would involve high losses of bentonite slurry and the risk that oversized boulders accumulated at the foot of the cofferdam severely hamper trench excavation.

The rockfill as well as the sand and gravel embankment dumped into the water will be very loose. The Contractor shall design in detail the measures necessary to stabilise the submerged cofferdam slopes. Sufficient stability could be achieved by deep underwater compaction of sand and gravel fill, sufficiently flat slope inclinations or an additional rockfill bund at the upstream toe. In any case, a surface protection in the river section will be placed.

When a dry surface has been reached the surface and the subsequent layers have to be compacted by a heavy drum roller. The cofferdam crests shall be stabilised by rockfill or by sand and gravel fill reinforced with tension fabrics to bear the load of the heavy trenching equipment.

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Two criteria have to be applied for determining the crest elevation of the cofferdams:

(i) The cofferdams must be safe against overtopping during floods with a reoccurrence interval of up to 100 years (2,900 m³/s) corresponding to a water level of 62.85 m asl. With a freeboard of 65 cm for wave run-up and wind-setup, and a safety margin of 0.5 m, the minimum crest elevation for being safe against overtopping is at 64.00 m asl.

(ii) The crest must be at sufficient elevation to provide the necessary head of the slurry during trench excavation for cut-off wall construction. The slurry weight exceeding the unit weight of water shall be treated as factor of safety for the trench stability. Then, the required slurry-level in the open trench is 64.50 m asl. A slurry level of 64.50 m asl can be achieved from a working platform of 65.00 m asl without raising the guidewall (see **Section 7.4**).

Therefore, the minimum crest elevation is 65.0 m asl.

The rockfill has to be of sufficient maximum grain size to build up an embankment in the river. Finer rockfill at the upstream face is desirable to form a transition to the sand and gravel fill. The rockfill can be of low strength but it shall not be sensitive to weathering under atmospheric conditions or under saturation and drying cycles.

Pleistocene sand and gravel from excavations can serve as sand and gravel fill for the cofferdams. The Holocene silt and clay is unsuitable to be incorporated into the cofferdam by dumping. A defined filter layer between rockfill and sand/gravel fill is not required, since some transport of sand/gravel into the rockfill voids would even be desired. This will not lead to erosion, since the construction pit will be dewatered only after finalization of the cut-off wall. Dewatering of the construction pit will cause settlements of the cofferdam embankment inside the cut-off wall.

The inclinations of submerged cofferdam slopes placed by end-dumping would appear in a natural inclination without allowing for a sufficient factor of safety against slope failure. Before major water level changes or equipment loads occur, the cofferdam slopes have therefore to be flattened to the following maximum inclinations:

- Rockfill: 1 : 2.0
- Sand and Gravel fill towards the river: 1 : 3.0
- Sand and Gravel fill towards the construction pit: 1 : 2.5

The results of the slope stability calculations are included in **Appendix 7-1**.

The cofferdam crest has to be of sufficient width to allow the equipment for construction of the slurry trench cut-off wall to operate and simultaneously a truck to pass. Ramps on the inner slope of the embankment to provide access to the construction pit will be made in the course of excavation according to the requirements of the contractor.

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### 7.2.3 Permanent Closure Embankment and Diversion Canal Backfill

The same principles of construction as for the cofferdam will be applied for the closure dam through the diversion canal. This involves initial canal closure by placing rockfill from barges or by end-dumping rockfill, extension by an upstream sand and gravel fill and subsequent sealing with a slurry trench cut-off wall.

The rockfill for initial closure of the diversion canal will be placed onto the existing riprap protection, so that hydraulic filter stability is ensured. At the cut-off wall location and further upstream where sand and gravel will be placed, the riprap in the diversion canal has to be removed to prevent direct seepage from the headpond.

Backfilling will extend over the entire upstream part of the diversion canal. In order to stabilize the submerged part of the sandfill slope, a rockfill bund will be placed along the upstream shoreline of the backfill. The excavated riprap from the diversion canal protection could be incorporated in that bund.

For underwater placement, only sand and gravel with a fines content of less than 10% is permissible. When the embankment crest has reached an elevation of at least two meters above the water level, deep vibro-compaction of the fill has to be performed. The expected settlements during vibro-compaction are in the order of 50 to 80 cm. The required density should be at least medium dense to exclude any liquefaction potential of the submerged material and excessive settlements over the long term.

Subsequent placing of material up to the final terrain elevation will comprise sand and gravel in the location of the administration compound and access roads. Silty material with the organic layer on top will be placed in the remaining area that shall be reclaimed for agriculture. The results of the slope stability calculations are included in **Appendix 7-1**.

The cut-off wall under the barrage structures continues along that axis for some 330 m up to the left bank of the former diversion canal. With this arrangement, seepage bypassing and under-streaming the cut-off wall results in an average exit gradient in the fine sand of $I_{average} = 0.02$ with a maximum of $I_{max} = 0.04$. This is far below the critical gradient of approximately 0.2.

### 7.2.4 Land Reclamation

The requirements for redistribution of the land to farmers and for implementation of the irrigation and drainage infrastructure will govern planning of the land reclamation. However, land can only be made ready for redistribution after finalisation of earthworks. This is to provide sufficient space for intermediate stockpiling and material processing facilities for the contractor according to his requirements.

The development of land reclamation has to take into account the variety of soil material to be incorporated in the reclaimed area. The material comprises agricultural soil, Holocene silt and clay, Pleistocene sand, gravel, and probably cobbles and boulders, and rockfill and plastic concrete debris from removal of the cofferdams and the cut-off wall. When placing the materials the following has to be considered:

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(i) Placing of material has to be performed such, that the coarse material like rockfill is placed deep under the surface and as far as possible in the downstream section.

(ii) Significant layer boundaries must be waived by forming transition zones of sufficiently mixed material in order to avoid erosion and suffusion.

(iii) Prior to placement of agricultural soil, the top 1.0 m thick layer of backfill material should comprise medium textured permeable material.

(iv) Agricultural soil which has been stockpiled separately can be placed to a minimum thickness of 0.9 m.

## 7.2.5 Backfill at Concrete Structures

For backfill of concrete structures, Pleistocene sand and gravel from excavations with a fines content of less than 10% shall be used. The material shall be placed in layers of 40 cm or less and be compacted by dynamic drum roller. The compliance of material properties and performed compaction with the technical specifications has to be checked by soil mechanic field laboratory testing.

## 7.2.6 Stability of Slopes

The applicable standards to calculate slope stability are DIN 1054 and DIN 4084.

The required factors of safety for slope stability for embankments are specified in DIN 4084 and are given in **Table 7.1** assuming as calculation procedure the slice method. Loads and load combinations to be considered are given in **Tables 7.2, 7.3** and **7.4**.

### Table 7.1: Required Factors of Safety for Slope Stability

|   | Load Case | Required FoS |
|---|-----------|--------------|
| I | Normal | 1.4 |
| II | Unusual | 1.3 |
| III | Exceptional | 1.2 |

The following embankments are subject to slope stability calculations:

A  - Diversion Canal Slope

B  - Right Bank Closure Dyke

C  - Diversion Canal Closure

D  - Construction Pit (Sample Calculation)

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**Table 7.2: Summary of Loads**

| No. | Kind of Load | Magnitude |
|---|---|---|
| 1 | Deadload | Bulk unit weight<br>Total unit weight |
| 2.1 | Range of river water levels before impoundment | 57.90 to 62.50 m asl |
| 2.2 | Range of river water levels after impoundment   upstream:<br>downstream: | 65.40 to 67.31 m asl<br>57.06 to 66.46 m asl |
| 3 | Range of water levels inside construction pit | 38.0 to 62.50 m asl |
| 4.1 | Road Traffic Load | 600 kN |
| 4.2 | Heavy Construction Traffic Load | 1,200 kN |
| 5 | Earthquake | 0.06 g horizontal<br>0.04 g vertical |
| 6 | Seepage into construction pit | inclined phreatic line |

**Table 7.3: Summary of Load Combinations and required Factors of Safety**

| No. | Load Combination | | Required Factor of Safety |
|---|---|---|---|
| | **Before Impoundment** | **Type** | |
| I | 1 + 2.1 + 4.1 | normal | 1.4 |
| Is | 1 + 2.1 + 4.1 + 5 | exceptional | 1.2 |
| II | 1 + 2.1 + 4.2 | unusual | 1.3 |
| IIs | 1 + 2.1 + 4.2 + 5 | exceptional | 1.2 |
| III | 1 + 3 + 4.2 | unusual | 1.3 |
| IIIs | 1 + 3 + 4.2 + 5 | exceptional | 1.2 |
| IV | 1 + 3 + 4.2 + 6 | exceptional | 1.2 |
| | **After Impoundment** | **Type** | |
| V | 1 + 2.2 + 4.1 | normal | 1.4 |
| Vs | 1 + 2.2 + 4.1 + 5 | exceptional | 1.2 |
| VI | 1 + 2.2 + 4.2 | unusual | 1.3 |

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## Table 7.4: Relevant Load Combinations for Embankment Stability Calculation

| | Embankment | | Relevant Load Combinations: | |
|---|---|---|---|---|
| | | | Before Impoundment | After Impoundment |
| A | Diversion canal slopes | | I, Is II, IIs | |
| B | Right bank closure dyke | | I, Is II, IIs | V, Vs VI |
| C | Diversion canal closure | upstream slope: | | V, Vs |
| | | downstream slope: | | V, Vs VI |
| D | Construction pit | | III, IIIs IV | |

With the above load combinations, the calculations were performed as detailed in **Appendix 7-1**, resulting in the factors of safety given in **Table 7.5**.

## Table 7.5: Calculated Factors of Safety of Slope Stability

| | Embankment | | Load Combination | Calculated (Required) Factor of safety |
|---|---|---|---|---|
| A | Diversion canal slope | | I | 1.81   (1.40) |
| | | | Is | 1.41   (1.20) |
| | | | II | 1.77   (1.30) |
| | | | IIs | 1.41   (1.20) |
| B | Right bank closure dyke | | I | covered by FoS in II |
| | | | Is | covered by FoS in IIs |
| | | | II | 1.92   (1.30) |
| | | | IIs | 1.57   (1.20) |
| | | | V | covered by FoS in IIs |
| | | | Vs | 1.45   (1.20) |
| | | | VI | 2.14   (1.30) |
| C | Diversion canal closure | upstream slope: | V | covered by A-I |
| | | | Vs | covered by A-Is |
| | | downstream slope: | V | covered by FoS in VII |
| | | | Vs | 1.61   (1.20) |
| | | | VI | 1.93   (1.30) |
| D | Construction pit | | III | 1.54   (1.30) |
| | | | IIIs | 1.22   (1.20) |
| | | | IV | 1.31   (1.20) |

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## 7.3    Riverbed Protection

The distribution of the different types of riverbed protection are shown on **Album No. 22 of Volume 3**. Details of the protections of riverbed and slopes are given in **Album Nos. 21, 24, 25, 26 and 27**.

### 7.3.1    Principles of Design

Riverbed protection types have been defined to achieve a sustainable protection of the bed and the banks of the river. Failure mechanisms of riprap are related either to erosion of individual particles from the surface or from washing out of underlying filter material and/or subsoil. In general the riverbed protection (types I to VI) is made up of:

- a top layer of riprap (types R1 to R6 , see **Table 7.6**) characterised by its mean diameter $D_{50}$,
- one or two transition layers of smaller grain sizes below, or even no transition layer, and
- a geotextile filter on top of the original riverbed .

At the barrage site, the riverbed material of the Nile is characterised by a mean diameter $D_{50} = 0.23$ mm and an uniformity coefficient of $U = D_{60}/D_{10} = 2.1$. At the slopes even silt may be encountered. In order to achieve an economic bed protection and effective filter or transition design for such fine material, geotextiles are used for the lowest layer on top of the sand at all permanent structures. On the geotextile, a gravel layer is placed for its protection against sharp edges of larger material when sunk.

### 7.3.2    Design of Riprap Top Protection Layers

The riprap types will be finally specified by their gradation of stones, the weight of which is derived from the Esaweia material with a density of 2,200 kg/m$^3$. The designed works themselves are based on size related data derived from the Esaweia material, which also was used in the model tests.

The optimum grain size distribution of the riprap material which acts as the protection layer is achieved for a ratio of $D_{85}/D_{15}{}^* = 3.0$ (equal to $U = D_{60}/D_{10} = 1.73$). This grain size distribution allows for an optimum interlocking of the individual stones [6][**]. In the relevant literature [6][**] similar optimum grain size distributions are given, e.g.

- $D_{84}/D_{16}$  > 1.96 - 2.56  (GÜNTER, 1971)
- $D_{84}/D_{16}$  > 1.5  (CHIN, 1985)
- $D_{90}/D_{50}$  > 1.55 (SCHÖBERL, 1981)

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[*]    $D_{15}$ is the size at which 15 percent of the total particles are smaller (the percentage is by weight). The $D_{85}$ size is defined correspondingly.

[**]    References see **Chapter 5**

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According to the filter stability criteria of WITTMANN [6][**], a higher uniformity coefficient U of the riprap permits a reduced thickness of the riprap layer but reduces also the interlocking effects. In the tender design, for riprap types R1 to R5 preference is given to optimum interlocking of the individual stones and, hence, a uniformity coefficient of U = 1.7 is maintained.

For bed protection type VI the riprap type R6 is used without geotextile. This requires that filter stability between river bottom soil and riprap type R6 is achieved. In order to reduce the required thickness (**Figure 7.1**) of the layer a higher uniformity coefficient of U = 5 is required.

Criteria for stability against erosion and filter stability selected and used for definition of grading and the required thickness of the riprap layer are different for permanent riverbed protection and temporary protected areas, such as the diversion canal with riprap type R6.

The risk of failure of riprap at permanent structures shall be almost zero. For eliminating the risks, no primary erosion is allowed for the permanent riprap protection works. It shall be prevented by the selection of larger size of stones but of a rather limited thickness of the bed protection layers. This approach is even more applicable when taking into account the difficulties in access to place the bed protection, in particular downstream of powerhouse and sluiceway, during operation of the barrage. Any repair works would have negative economic side effects as such.

For temporary works the thickness of the riprap (type R6) shall not be less than the depth of primary erosion plus the required thickness of the armoured riprap layer (PAINTAL, 1971). A limited erosion of small and medium sized stones in order to achieve armouring of the top riprap layer is allowed.

The thickness of the riprap follows the recommendations on common construction techniques and stability against erosion:

- PIANC Bulletin 1987 $\quad\quad\quad D_{Riprap} > 2 \cdot D_{50}$
- OGRIS, 1975 $\quad\quad\quad\quad\quad D_{Riprap} > 2 \cdot D_m$

General field experience has shown that the riprap layer should be 1½ times, or more [8][**] than the largest particles which is in line with the above criteria ($D_{90} = 1.5 \cdot D_{50}$ and $D_{Riprap} = 1.5 \cdot 1.5 = 2.2 \cdot D_{50}$). Accordingly, the thickness of the top riprap layer was defined to be in the order of $2.2 \cdot D_{50}$, or more.

For economic reasons the thickness of bed protection shall be a minimum. However, to achieve filter stability, the thickness of the riprap shall not be smaller than the required filtration length, i.e. $L_f > 10 D$; $D = f(U, D_{50})$ (WITTMANN, 1982, [6][**]) as shown in **Figure 7.1**. For the riprap type R6 with a coefficient of uniformity of U = 5.0 the layer thickness results with the filter stability criteria of $10 \cdot D = 10 \cdot (0.16 \dots 0.40) \cdot D_{50}$ in a minimum thickness of between 0.16 and 0.40 m. As a result, the riprap thickness was selected to 0.7 m, in order to be sufficiently safe, even after reduction by some 0.2 m due to primary erosion or sinking of the dumped particles into the sand of the river bottom.

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[**]  References see **Chapter 5**

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For riprap types R1 to R5, which have a uniformity coefficient of $U = 1.7$, the required thickness of the riprap layer to achieve a stable filter results in at least $6 \cdot D_{50}$, which is uneconomically. In order to reduce the thickness of the layer and still to achieve sufficient filter stability a geotextile was incorporated as the lowest part of the bed protection types I to V.

For bed protection type VI the riprap type R6 is used without geotextile. This requires that filter stability between river bottom and riprap type R6 is achieved. In order to reduce the required thickness (Figure 7.1) a higher riprap with a higher uniformity coefficient of $U = 5$ is required.

Criteria for stability against erosion and filter stability are developed for the definition of grading and the required thickness of the riprap layer are different for permanent and diverted protection and temporary protected structures as shown in the divison onut and riprap types R6.

The risk of failure of riprap at permanent structures should be almost zero. For eliminating the risks, no primary erosion is allowed for the entire bed protection works, it shall be prevented by the selection of larger stones and by a rather limited thickness of the bed protection layers. This approach is well justified especially taking into account the difficulties in access to, maintenance and operation. It is a matter of prudence and security, and especially, during operation of the damage. Any repair works would have negative economic side effects as such.



**Figure 7.1: Required Thickness of Protection Layer (WITTMANN)**

### 7.3.3 Design of Transition Layers

Wash out of transition or subsoil material due to turbulence induced pressure fluctuation or other hydraulic forces shall not be permitted. A geotextile and/or transition layers must be used to prevent internal erosion and suffusion. In all cases the lowest layer placed directly onto the geotextile should not exceed the type R4 in order not to exceed the admissible drop cone resistance and to damage the geotextile while dumping the stones from a barge onto the geotextile.

To achieve filter stability, local flow of pore water shall not be able to convey fine particles of granular materials or subsoil particles through the pores of the coarse particles of a transition or through those of the geotextiles.

The traditional design criterion requires a filter to be geometrically tight, which implies pore or opening sizes being too small to allow the finer grains to pass through. During recent years, criteria for geometrically open filters have been developed which, in many cases, produce a more economical design. The criteria for geometrically open filters are based on the principle that the hydraulic load must be below the critical value for initiation of erosion of the base material.

In the transitions design both principles are applied. The criteria for geometrically open filters are applied using the approach of CISTIN/ZIEMS [6]**. Based on the gradation of the top layer and the transition material arranged below, a maximum admissible ratio of $D_{50}$ of the top layer and the layer below is derived from **Figure 7.2**.

With the uniformity coefficients of the subsoil $U_S = 2.1$ (river Nile, $D_{50} = 0.23$ mm) and of one of the top layers $U_R = 1.7$ (riprap, $D_{50} = 0.10 \ldots 0.74$ m), the general need for a filter or transition layer was established by **Figure 7.2**. The actual ratio $D_R/D_T$ varies between 435 and 3,200, which is far above an admissible ratio of 10.

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** References see **Chapter 5**.

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Riprap types R4 to R6 and a gravel type G1 are applied as transition layers. With the uniformity coefficients U varying between 1.5 and 5.0, the $D_{50}$ ratio of top layer and base material may not exceed a value of between 6 and 10 as shown in **Table 7.6**. The actually selected ratios are maintained significantly below the permissible values.

**Table 7.6: Top and Transition Layers of Riprap**

| Top Layer | | | Transition Layer | | | Ratio | |
|---|---|---|---|---|---|---|---|
| Type | $D_{50}$ m | Uniformity | Type | $D_{50}$ m | Uniformity $(D_{60}/D_{10})$ | Permissible $D_{R,50}/D_{T,50}$ | Actual $D_{R,50}/D_{T,50}$ |
| R1 | 0.74 | 1.5 | R4 | 0.26 | 1.7 | 6 | 2.9 |
| R2 | 0.54 | 1.5 | R5 | 0.17 | 1.7 | 6 | 3.2 |
| R3 | 0.39 | 1.7 | R6 | 0.10 | 5.0 | 10 | 3.9 |
| R4 | 0.26 | 1.7 | G1 | 0.06 | 5.0 | 9 | 4.4 |
| R5 | 0.17 | 1.7 | G1 | 0.06 | 5.0 | 9 | 2.9 |
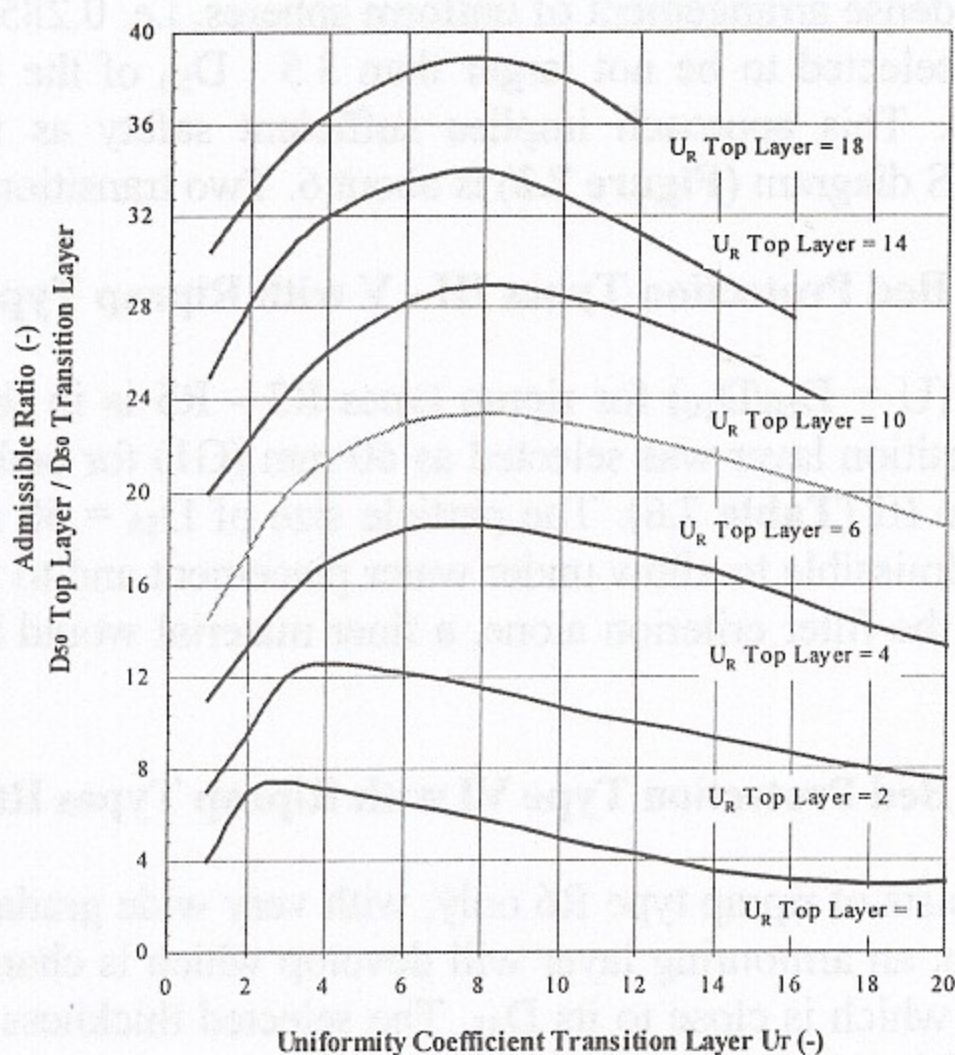


**Figure 7.2: Diagram of CISTIN/ZIEMS, Permissible Diameter of Top Layer and Transition Layer Material**

A summary of the riverbed protection types, their constitution and the selected transition layers is given in **Table 7.7**. Details on the individual transition layers are given in the paragraphs below.

The hydraulic model tests were carried out using limestone from Esaweia with a dry density of 2,200 kg/m³, which is the basis of above and below tables. In case the riprap material selected for riverbed protection will have a different density, the mean diameter of the riprap is to be adjusted. The scaling factor to determine the particle size of the riprap material with different density is given by the ration of the cubic root of the density for model riprap and for the different prototype riprap. In particular for riprap types R1 and R2, application higher density and strength material is recommended.

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### Table 7.7: Riverbed Protection Types (Riprap Density of 2,200 kg/m³)

| Bed Protection Type | Riprap $D_{50}$ in m | Thickness in m | Transition 1 $D_{50}$ in m | Thickness in m | Transition 2 $D_{50}$ in m | Thickness in m | Total Thickness in m | Geo-textile |
|---|---|---|---|---|---|---|---|---|
| I | 0.74 | 1.60 | 0.26 | 0.60 | 0.06 | 0.30 | 2.50 | Yes |
| II | 0.57 | 1.20 | 0.17 | 0.40 | 0.06 | 0.30 | 1.90 | Yes |
| III | 0.39 | 1.00 | 0.10 | 0.30 | - | - | 1.30 | Yes |
| IV | 0.26 | 0.80 | 0.06 | 0.30 | - | - | 1.10 | Yes |
| V | 0.17 | 0.60 | 0.06 | 0.30 | - | - | 0.90 | Yes |
| VI | 0.10 | 0.70 | - | - | - | - | 0.70 | No |

### (1) Transition Design for Bed Protection Types I and II with Riprap Types R1 and R2

For the coarse riprap material (R1, R2) the mean diameter of the upper transition material was defined to achieve tight geometrical conditions. The design criterion was selected to be the mean value of the most and least dense arrangement of uniform spheres, i.e. $0.285 \cdot D_{50}$. The top layer (riprap) material was thus selected to be not larger than $3.5 \cdot D_{50}$ of the bottom or transition material (riprap or gravel). This approach implies sufficient safety as the maximum ratio according to CISTIN/ZIEMS diagram (**Figure 7.2**) is about 6. Two transition layers are required.

### (2) Transition Design for Bed Protection Types III - V with Riprap Types R3 - R5

The uniformity coefficient ($U = D_{60}/D_{10}$) for riprap types R3 - R5 is in the order of 1.7. The particle size $D_{50}$ for the transition layer was selected as 60 mm (G1) for bed protection types IV and V and 100 mm for type III (**Table 7.6**). The particle size of $D_{50} = 60$ mm was selected as minimum mean diameter admissible to allow under water placement and to prevent segregation, although for satisfaction of the filter criterion alone, a finer material would have been sufficient for type V.

### (3) Transition Design for Bed Protection Type VI with Riprap Types R6 ($D_{50} = 10$ cm)

Bed protection type VI consists of riprap type R6 only, with very wide gradation (U = 4.5 - 5.0). On the surface of this riprap, an armouring layer will develop which is characterised by a mean diameter of some 170 mm, which is close to its $D_{85}$. The selected thickness of 70 cm considers the primary erosion. An additional geotextile is not necessary.

### (4) Transition by Geotextiles

Permeable geotextiles made from artificial fibres are used in conjunction with soil or rock as an integral part of the riverbed protection types I to V. They are used as filter membranes, and preference is given to for the purposes of this project non-woven fabric. The maximum standard width of geotextiles available in rolls both woven and non-woven fabrics is 5 to 5.5 m. Depending on the density per unit area, the length of the rolled material is in the range from 50 to 200 m. Geotextiles can overlap, be sewed together or be welded to larger areas. In general, the most commonly used mass of non-woven geotextiles lies in the range of 100 to 300 g/m².

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The pore sizes of the geotextile must be sufficiently small to prevent erosion from the underlying material of the riverbed or of the riverbanks. The relevant property of a geotextile is the "effective opening size" $D_W$, which is determined by sieving, using the geotextile as the sieve. From the result of sieving $D_W$ equals the grain size diameter from which only 10% have passed the geotextile.

The geotextile is subject to tensile stress during installation, and subject to perforation stresses when coarse material is placed on it, either in the dry or dropped under water.

The specified products "Terrafix 813 and Terrafix 609 or similar", shall have a tensile strength of 12 KN/m and a drop cone resistance of 1,200 Nm allowing to dump particles with a weight of up to 165 kg under water. The "Terrafix 813" is filter resistant against all soil types, while the "Terrafix 609" is suitable for the Pleistocene sand, but not for the silt.

## (5) Transitions from the Riprap Protected to the Unprotected Riverbed

Downstream of the transition from the protection to the original riverbed, scouring is likely to occur. Discontinuities in riverbed roughness and bed elevation at the transition will cause some change of shear stress and turbulence which may contribute to scouring. This process is intensified by the concentration of flow during typical modes of operation of the New Barrage compared to natural river flow conditions. By the conditions of the natural riverbed, there is also a general but weak tendency to progressive downstream riverbed degradation, which results from the discrepancy between the sediment transport capacity of the flow and the limited sediment yield from the upstream.

With reasonable economic effort, scouring downstream of the protected riverbed cannot be avoided. Therefore, the objective of the transition design is to limit the extent of scouring as to have no impact on safe operation of the barrage and navigation. Retrogressive scour development leading to undermining of the protected portion of the riverbed must be prevented. Therefore, at the transition from the protected to the original riverbed, an additional reinforcement of the gravel layer on top of the geotextiles is provided.

Special geotextiles such as the specified "Tensar GM4 grids or similar approved", will be placed over the last 25 m of the downstream riverbed protection in order to achieve a reinforcement of the underlying gravel on top of the base geotextile and thus of the entire riverbed protection works.

Downstream of the barrage, the transition zone scour protection will comprise the following components, listed from top to base:

- Riprap of specified type, i.e. R3 or R4,
- Tensar GM4 grid (or approved similar),
- riprap type R6 ($D_{50}$ = 100 mm) layer of 30 cm,
- Tensar GM4 grid (or similar approved),
- gravel ($D_{50}$ = 60 mm) layer of 30 cm,
- geotextile.

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At the downstream end, a length of about 3 m of the base geotextile shall remain unpaved without the protection layer. This part will be able to follow the initial formation of a scour hole. The suggested stabilisation measures are sufficiently flexible to adapt to the developing shape of a scour hole and to prevent the riprap and gravel material from migrating down.

### (6) Reinforced Riverbed Protection Covering the Cut-off Wall Trench

The remainders of the cut-off wall along the downstream cofferdam crest alignment represents a protection against retrogressive scour development. Any scour development which unexpectedly could reach the downstream cut-off wall will be stopped as long as there exists a sustainable protection between cut-off wall and the riverbed protection works. With the aim to provide at that location for any unforeseen failure of the riprap additional safety, the riverbed protection above the cut-off wall will be reinforced by means of "Tensar GM4 grids or similar approved" over a width of about 8 m. The grid will be placed on top of the gravel transition layer.

### 7.3.4 Design of Slopes

In the design of the bank slopes and their protection, different conditions were to be considered for the river reaches upstream and downstream of the new barrage. The significant difference between these two river reaches is the depth of water and the possible variation of the water level, and thus the hydraulic impacts acting on the slopes. The water level variation in the headpond is rather limited, i.e. -0.5 m and +0.3 m under normal operation conditions and +1.5 m for the emergency flood including freeboard and wave action.

In contrast, the embankments in the river reach downstream of the new barrage will be subject to higher variations of the water level, i.e. seasonal variations between 57.90 and 61.90 m asl plus waves. The maximum water level during the emergency release may reach 66.40 m asl. The most frequent water level around the year, i.e. for a discharge of about 1,400 m³/s will be at 60.30 m asl. In this case the waves generated by wind or navigation will attack the flattest and thus most stable part of the embankment.

### (1)    Protection of Slope Toes

As a protection measure at the toe of the bank protection, reinforcement will be made by large mesh geotextile mats which can over take tensile stresses due to deformations under the base geotextile by scouring. "Tensar mattresses or similar approved" filled with gravel $D_{50} = 60$ mm (6.0 m · 2.0 m · 0.3 m) can be placed on the base geotextile. The thickness and composition of the bank protection will not be effected thereby.

### 7.3.5    Riverbed Protection by Placed Riprap

Conventional riverbed protection consists of dumped riprap with or without transition layers between the top layer and the subsoil. If the transition layers are thick and require additional excavation, then less costly types of riverbed protection which provide a similar degree of safety and durability, must be considered.

A typical location where riverbed protection by means different from dumped riprap would be of advantage, is the powerhouse exit bay area, foreseen to be protected by type R1.

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Individually placed riprap particles must be of better quality than the minimum suitable for dumped riprap. Placement must be such that, e.g. the placed riprap approaches good dry rubble in quality and appearance, in a more or less definite pattern with a minimum amount of voids. Stones of a flat, stratified nature should be placed with the principal bedding planes normal to the slope. Joints should be broken as much as possible, and joint openings to the underlying fill should be avoided by carefully arranging the various sizes of stones and closing the openings with small rock fragments. The thickness of placed riprap could be one half of the thickness required for dumped rock riprap [8][**].

The requirement for the riprap type which forms the upper layer of the bed protection type (I or II) may be reduced by one class, i.e. instead of conventional dumped riprap Type R1 individually placed riprap Type R2 can be used, e.g. downstream of the powerhouse. Individually placed riprap layers will have a thickness of not less than $1.5 \cdot D_{50}$ (**Table 7.8**). A modified riverbed protection type I* (or II*) with individually placed riprap type R2 as the top protection layer will be tested in the hydraulic model tests. Only when the results are available and have been assessed such replacement of dumped riprap types could be finally recommended.

### Table 7.8: Bed Protection Types for Placed Riprap (Density 2,200 kg/m³)

| Bed Protection Type | Riprap $D_{50}$ in m | Thickness in m | Transition 1 $D_{50}$ in m | Thickness in m | Transition 2 $D_{50}$ in m | Thickness in m | Total Thickness in m | Geo-textile |
|---|---|---|---|---|---|---|---|---|
| I | 0.74 | 1.60 | 0.26 | 0.60 | 0.06 | 0.30 | 2.50 | Yes |
| I* | 0.57 | 1.00 | 0.17 | 0.40 | 0.06 | 0.30 | 1.70 | Yes |
| II | 0.57 | 1.20 | 0.17 | 0.40 | 0.06 | 0.30 | 1.90 | Yes |
| II* | 0.39 | 0.70 | 0.10 | 0.30 | - | - | 1.00 | Yes |

## 7.4    Temporary and Permanent Slurry-Trench Cut-Off Walls

### (1)  Kind of Cut-off Walls

The most suitable technique for establishing a watertight diaphragm wall under the given geological conditions and the required dimensions of the wall is the slurry-trench technique. Two kinds of slurry-trench cut-off walls exist, which are the one-phase and the two-phase cut-off walls. In the one-phase technique, the slurry used for trench stabilisation remains in the trench after excavation for setting and forming the cut-off wall, while in the two-phase technique the slurry is replaced by concrete.

Two alternative excavation methods could be applied, which is excavation by grab or excavation by trench cutter. The very advanced techniques of monitoring and adjusting verticality during trench cutting favour the cutting technique. Moreover, excavation by trench cutter is seen more economic due to the quicker excavation speed.

The permanent cut-off walls under the structures, through the backfill and through the closure embankment will be two-phases slurry trench cut-off walls excavated by trench cutter. The kind of the temporary cut-off wall around the construction pit shall be determined and designed by the contractor.

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[**] References see **Chapter 5**

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## (2) Layout of Cut-off Walls

The permanent cut-off wall below the structures is connected to the temporary cut-off wall of the construction pit at both river banks. Along the powerhouse, sluiceway and navigation locks the cut-off wall penetrates into the clay layer.

As the clay layer shows a significant slope from east to west, the depths of the cut-off wall is some 45 m below surface besides the navigation locks and some 55 m below surface besides the powerhouse.

A lateral extension beyond the diaphragm of the construction pit at the right bank is not necessary. Various calculations (**Appendix 7-2**) have shown, that the increment of the flowpath by an extent of the cut-off wall into the right bank (some 100 m to 200 m) is to small compared to the existing flowpath of some 600 m. As a result, a significant improvement concerning the right bank exit gradients was not assessed.

The optimisation of the cut-off wall (**Appendix 7-2**) layout results in a 330 m long intersection through the closure embankment and powerhouse backfill as shown on **Album No. 70**. The temporary cut-off wall of the construction pit completely intersects into the clay layer. The top level of this wall is given with 66.00 m asl.

## (3) Dimensions

Three criteria have to be applied for determining the thickness of the cut-off wall:

(i)   Maintaining a minimum overlapping of adjacent panels at final depth.

(ii)  Ensure that the gradient in the cut-off wall is with safety less than permissible.

(iii) Stability of the cut-off wall under the expected load conditions.

In order to limit the gradient to less than 20 at a maximum head of some 9 m, a minimum overlap of 45 cm must be granted between adjacent panels. Inclination tolerances can realistically be achieved in the order of 0.3% at best. This results in a maximum relative displacement between adjacent panels of some 24 cm at a depth of 40 m. Hence, the required minimum nominal width of the temporary cut-off wall surrounding the construction pit is 69 cm. This results in a specified width of 80 cm to comply with common sizes of trench cutters.

For the temporary cut-off wall, a higher gradient of up to 50 might be accepted.

The cut-off wall under the permanent structures and through the backfill of the navigation lock will penetrate for 3 m into the silt and clay layer. Due to the expected depth of that layer of more than 55 m below surface on the left bank the bottom of the cut-off wall will be at 27 m asl over 150 m adjacent to the powerhouse abutment pier and at 39 m asl over the remaining 180 m. The exact depth of that layer and its continuous existence along the scheduled alignments of the cut-off walls must be explored by drilling prior to cut-off wall construction. The boreholes shall be located some 5 m off the alignment outside the construction pit with a spacing of not more than 50 m. On the cofferdams, these boreholes can be performed during dumping from an intermediate level. Along the alignment of the cut-off wall under the structures, the boreholes shall be located on the downstream side. The boreholes shall completely penetrate the clay layer and shall be deep enough to assess, that the cut-off wall in the respective location is properly tied into the clay layer and not in local clay lens. After finalisation of drilling, all boreholes have to be completely grouted.

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## (4) Properties

The upper part of the cut-off wall under the structures may be reinforced. The concrete mix for the temporary as well as for the permanent cut-off walls shall be designed by the contractor resulting in a plastic concrete with a high deformation modulus and low strength. The material properties shall be:

- Cut-off wall under the barrage structures, through the backfill and in the closure embankment:
    - Permissible Gradient: 20
    - Permeability: $< 10^{-8}$ m/s
    - Strength and deformation modulus suitable for the expected head up to 9 m.

- Cut-off wall around the construction pit:
    - Permissible Gradient : 50
    - Permeability: $< 10^{-8}$ m/s
    - Strength and deformation modulus suitable for the expected head up to 25 m.

The contractor shall perform the laboratory testing program on the raw materials and on samples of the plastic concrete in accordance with the technical specifications. The Employer/Consultant will crosscheck the contractor's testing results by his independent materials testing program.

## 7.5    Dewatering and Excavation of Construction Pit

Dewatering of the construction pit shall only commence after the cut-off wall is fully completed. Otherwise seepage, in particular through an unsealed downstream cofferdam, may result in erosion of the sandfill.

Dewatering will commence with initial pumping out the water basin in the construction pit and dewatering of the saturated in-situ soil. Pumping wells and pump sumps for dewatering the in-situ soil in the construction pit have to operate simultaneously with pumping out the water basin to avoid hydraulic fractures and erosion of slopes.

Drainage wells have to be arranged in sufficient quantity to release water pressure inside the sealed construction pit. The installation of drainage wells has to commence at sufficiently high elevation to avoid hydraulic fracture within foundation areas, since this would decrease the bearing capacity.

It will be the contractor's responsibility to place and operate the appropriate number of pumping wells, drainage wells and pump sumps at the most suitable locations to keep the phreatic line well below the foundation levels. Accordingly, he is obliged to observe the seepage conditions and to have sufficient stand-by equipment for wells and pumps to control unexpected and unfavourable events.

It must also be assured that before approaching by excavation the lower levels of foundation, the piezometric pressure below the clay layer is monitored. This is necessary to control uplift

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conditions which might result from artesian pressure induced from the Old Barrage headpond to lower aquifers.

Calculations given in **Appendix 7.3 Construction Pit – Safety against Uplift** have indicated that under certain conditions of excavation the factor of safety against uplift enter a critical range.

The effort for dewatering the construction pit mainly depends on the quality of the cut-off wall around the construction pit. With a tight cut-off wall (permeability coefficient $10^{-8}$ m/s) and a continuous silt and clay layer at depth (permeability coefficient $10^{-8}$ m/s), the seepage rate will be small. Local seepage into the pit due to imperfections during construction of the cut-off wall will result in a reduced overall permeability of some $10^{-7}$ m/s. For that case and a silt and clay layer assumed as continuous, a basic discharge of some 250 to 300 l/s mainly through the cut-off wall can be expected. The additional discharge resulting from a possible gap in the silt and clay layer over some 10 % of the construction pit area was calculated in the order of magnitude of further 100 l/s.

A network of monitoring piezometers at the construction pit is essential for controlling the success of the dewatering measures and, if necessary for the design of remedial measures, where necessary. For this purpose, the boreholes to be drilled along the alignment of the cut-off wall shall be equipped with piezometer pipes, located above and below the silt and clay layer. Another series of piezometers shall be installed inside the construction pit at an elevation above the silt and clay layer.

## 7.6    Preparation of Foundation for Concrete Structures

### 7.6.1    Foundation Characteristics

The structural components of the barrage will be found on varying levels as outlined below:

- Powerhouse Abutment Pier:    38.00 to 42.40 m asl
- Powerhouse:    38.00 to 42.40 m asl
- Intermediate Pier:    38.00 to 42.40 m asl
- Sluiceway:    41.90 to 47.50 m asl
- Sluiceway Abutment Pier:    43.50 m asl
- Navigation Locks:    43.50 to 50.50 m asl

All foundations are stiff and the structural behaviour will be of the rigid block foundation type with well-distributed foundation pressures. The foundation of sluiceway and navigation lock will also be by rigid blocks, however the foundation pressure under the sluiceway will be somewhat more concentrated under the piers due to the considerable span of 17 m. Similarly, the earthfill between the two navigation locks will rise the foundation pressure between the navigation locks.

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The weight of the concrete (including sandfill, where applicable) distributed evenly over the foundation area would result in the following average pressures:

- Powerhouse Abutment Pier:    $420 \text{ kN/m}^2$
- Powerhouse:    $312 \text{ kN/m}^2$
- Intermediate Pier:    $324 \text{ kN/m}^2$
- Sluiceway:    $178 \text{ kN/m}^2$
- Sluiceway Abutment Pier:    $309 \text{ kN/m}^2$
- Navigation Lock:    $179 \text{ kN/m}^2$ ($318 \text{ kN/m}^2$ at filling system)

## 7.6.2 Expected Foundation Conditions

### (1) Powerhouse

In the foundation area of the powerhouse, a layer consisting of sand, gravel and probably cobbles and boulders was encountered between 37.55 and 41.50 m asl in borehole (BH) 26 (**Volume 4, Figures 2.10** and **2.11**). Hence, foundation of the powerhouse will be within and also above that layer. The intermediate pier and the powerhouse abutment pier will also be found within that layer. Due to the large foundation area of the structures and the irregular distribution of gravel and stones layers, an inhomogeneous foundation surface for the powerhouse and powerhouse piers consisting of various soil types has to be expected.

Five standard-penetration-tests executed in the underlying 10 m of sand below 37.55 m asl resulted in 26, 29, 69, 58 and 64 blows/30cm, respectively. Below up to final depth, the number of blows always exceeded 62. According to the number of blows, the sand is medium dense to dense between 37.55 and 34.00 m asl and very dense below 34.00 m asl.

### (2) Sluiceway

According to BH 9 (**Volume 4, Figures 2.7, 2.9** and **2.10**), the foundation area of the sluiceway will be right above a sand and gravel layer probably containing some cobbles and boulders. Due to the irregular geological stratification, and the large foundation area, a variety of soil types have to be expected in the sluiceway foundation level.

Six standard-penetration-tests executed under the gravel and cobbles layer at 41 m asl resulted in 30, 34, 29, 36, 38 and 39 blows/30cm, respectively, indicating medium dense to dense or dense material. Below up to final depth, the number of blows always exceeded 69, indicating very dense material.

### (3) Navigation Locks

In the foundation area of the navigation locks, medium to fine sand locally with some coarse sand or fine gravel was encountered in boreholes 5 and 35 (**Volume 4, Figures 2.9, 2.13** and **2.14**). Seven standard-penetration-tests executed between 50.5 and 46.0 m asl resulted in 14, 17, 18, 19, 20, 28 and 29 blows/30cm, respectively, indicating medium dense material. Subsequent tests in BH 35 resulted in a continuous increase in number of blows, indicating at least dense material. The same applies to BH 5 with one exemption between 36 and 42 m asl where only medium dense material was encountered.

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## (4) Administration Compound

The administration buildings and workshops are located on the backfilled material in the upper part of the diversion canal at elevation 69 m asl. The bottom zone dumped under water will be sufficiently compacted by vibroflotation. The subsequent placing of material under dry conditions could be performed in compacted layers of sand and gravel.

### 7.6.3 Treatment of Foundation Areas

### (1) Barrage Components

The natural density of the sand and sand-gravel mixtures encountered under the foundation of powerhouse, sluiceway and piers can be expected as sufficient for bearing the loads under acceptable settlements, if the foundation work is performed properly.

With the loads and assumed soil conditions as outlined before, the settlements of the powerhouse and the abutment and intermediate piers can be expected in the order of magnitude between 3 and 6 cm and for the sluiceway between 2 and 4 cm. The foundation conditions must not be assumed homogenous under the large foundation areas however, differential settlements between structural units of the sluiceway caused by a varying stratification can be assumed to be less than 1 cm. The settlements of the sluiceway abutment pier and the navigation lock filling and emptying system are expected between 3 and 5 cm.

The natural density of the soil in the foundation area of the upstream navigation lock chambers is only medium. However, drawdown of groundwater inside the construction pit to a level up to 13 m below the navigation lock foundation will have a considerable preloading effect. With the structural loads and assumed soil conditions as outlined before, the settlements of the upstream navigation lock blocks can be expected in the order of magnitude between 2 and 4 cm. Differential settlements between the blocks of the navigation lock caused by a varying stratification can be assumed far less than 1 cm.

The settlements will mostly occur immediately with load application or within a few days thereafter.

A continuous settlement monitoring program shall be implemented at site and be evaluated according to the records of load application.

A precondition for settlements being limited to a permissible range is a proper performance of foundation works comprising:

(i) All foundation areas must be sufficiently dewatered in advance, before the final excavation lines are reached in the following two respects:

- Soil must be dewatered for at least 0.5 m below deepest excavation level.
- Soil must be released from confined, semi-confined or artesian waterpressure for the entire depth up to the silt and clay layer.

(ii) Excavation of in-situ soil has to be terminated at least one meter above the final foundation level. From this level or higher the cut-off wall construction will be performed, where necessary. Excavation of the final lines must be performed backwards without manoeuvring

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(iii) For the case that soil unsuitable for foundation is encountered in the final excavation area, the respective portion has to be further excavated and replaced by a well graded gravel-sand mixture. Unsuitable soil for foundation may be silt lenses, uniform gravel that cannot be compacted or gravel and boulders that do not form a proper soil matrix for placing concrete due to an insufficient content of gravel and sand.

(iv) Proper preparation and compaction of inclined foundation areas may be difficult due to insufficient stability of inhomogeneous slopes under the weight of compaction equipment. Such areas have to be over-excavated for at least 2 m of width. Well-graded gravel-sand-mixture has to be placed and compacted in horizontal layers at least 0.5 m wider than finally required. Final slope trimming will then be performed through the fill material.

## (2) Diversion Canal

Settlements of that part of the diversion canal backfill that is placed above the water level under dry conditions would be negligible if the material is placed in layers properly compacted. However, the settlements of the very loose 7 to 9 m thick layer that was dumped under water can be expected in an order of magnitude between 20 and 40 cm. Dynamic surface compaction once the embankment has risen above the water level would have no significant compaction effect on the underlying loose embankment. The application of the static weight of the embankment placed under dry conditions would cause some settlements but still the underwater embankment is of loose density and future fluctuations of the headpond level would repeatedly cause settlements.

Therefore, deep vibro-compaction of the fill has to be performed when the embankment crest has reached an elevation of at least two meters above the water level. Alternatively, the entire sequence can be compacted with deep vibrators, allowing a much quicker filling up to the final elevation without conventional compaction.

The deep vibro-technique shall be applied by compacting the soil with deep vibrators only and without adding further material. The expected settlements during vibro-compaction are in the order of 50 to 80 cm. Trial compaction at commencement of the works will be necessary to determine the most appropriate vibrator characteristics such as length and diameter, speed and amplitude. The permissible spacing of the grid is expected between some 2.2 and 2.8 m, however this will be determined by evaluation of trial compaction with various spacings and cone penetration testing in the center point of three compaction points. The required cone penetration resistance is 10 MPa. Cone penetration testing shall be further performed during the work to assure the quality of the compaction.

## 7.7    Excavations and Handling of Materials

All materials that are excavated shall be reused for other purposes so that no disposal of material within or beyond the area of the construction site is required unless the material is artificially contaminated or results from clearing, grubbing or stripping operations. This requires selective excavation to comply with the material requirements as outlined in **Volume 4, Section 4.1** and in the Technical Specifications of the Tender Documents. In addition, materials that cannot be borrowed from excavations in sufficient quantity or quality have to be purchased from external sources. Such materials are riprap, rockfill (partly) and concrete aggregates and transition layers (unless processed on site).

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Although it will be the aim of the contractor to avoid double handling of material but place excavated material without stockpiling at another final location, some intermediate stockpiling will be unavoidable. This refers mainly but not limited to:

a)  agricultural soil until reclamation areas have been established

b)  rockfill for cofferdams to achieve required dumping rate for final closure

c)  suitable material for backfilling of structures

d)  suitable material for backfilling of diversion canal

e)  rockfill excavated from cofferdams for reuse in closure dam

In general, large stockpile areas can be located in the flood channel, while the stockpile area on the left bank is limited to some 240,000 m$^2$. Any facilities for processing of materials, if desired by the contractor, shall be located in the flood channel.

According to the results of the geotechnical investigations as described in **Volume 4**, the expected materials and quantities (apart from agricultural topsoil where applicable) for the main excavation areas are given in **Table 7.12**.

**Table 7.12: Excavation Areas - Materials and Quantities**

| Location | Level | Prevailing Material | Quantities mio. m$^3$ |
|---|---|---|---|
| a) Diversion Canal | above 60 m asl (average) | sandy, clayey SILT, find SAND | 1.6 |
| | below 60 m asl (average) | SAND | 1.8 |
| b) Right Bank | above 58 m asl (average) | sandy, clayey SILT | 1.1 |
| | below 58 m asl (average) | fine SAND | 0.7 |
| c) Cut-off Walls | all depths | SILT/SAND/GRAVEL | 0.1 |
| d) Construction Pit | above 58 m asl (average) | sandy, clayey SILT | 0.8 |
| | below 58 m asl (average) | SAND, GRAVEL, some STONES | 1.1 |
| e) Cofferdams | as placed | ROCKFILL | 0.25 |
| | as placed | SAND, GRAVEL | 0.15 |
| f) Riprap Diversion Canal | as placed | RIPRAP | 0.15 |

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# APPENDICES

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# Appendix 7-1

# Results of Slope Stability Calculations

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## Table A7-1.1: Summary of Loads

| No. | Kind of Load | | Magnitude |
|---|---|---|---|
| 1 | Deadload | | Bulk unit weight<br>Total unit weight |
| 2.1 | Range of river waterlevels before impoundment | | 57.90 to 62.50 m asl |
| 2.2 | Range of river waterlevels after impoundment | upstream:<br>downstream: | 65.40 to 67.31 m asl<br>57.06 to 66.46 m asl |
| 3 | Range of water levels inside construction pit | | 38.0 to 62.50 m asl |
| 4.1 | Road Traffic Load | | 600 kN |
| 4.2 | Heavy Construction Traffic Load | | 1,200 kN |
| 5 | Earthquake | | 0.06 g horizontal<br>0.04 g vertical |
| 6 | Seepage into construction pit | | inclined phreatic line |

## Table A7-1.2: Summary of Load Combinations and required Factors of Safety

| No. | Load Combination | | Required Factor of Safety |
|---|---|---|---|
| | *Before Impoundment* | *Type* | |
| I | 1 + 2.1 + 4.1 | normal | 1.4 |
| Is | 1 + 2.1 + 4.1 + 5 | exceptional | 1.2 |
| II | 1 + 2.1 + 4.2 | unusual | 1.3 |
| IIs | 1 + 2.1 + 4.2 + 5 | exceptional | 1.2 |
| III | 1 + 3 + 4.2 | unusual | 1.3 |
| IIIs | 1 + 3 + 4.2 + 5 | exceptional | 1.2 |
| IV | 1 + 3 + 4.2 + 6 | exceptional | 1.2 |
| | *After Impoundment* | *Type* | |
| V | 1 + 2.2 + 4.1 | normal | 1.4 |
| Vs | 1 + 2.2 + 4.1 + 5 | exceptional | 1.2 |
| VI | 1 + 2.2 + 4.2 | unusual | 1.3 |

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## Table A7-1.3: Calculated Factors of Safety of Slope Stability

| | Embankment | Load Combination | Calculated (Required) Factor of safety |
|---|---|---|---|
| A | Diversion canal slope | I | 1.81    (1.40) |
| | | Is | 1.41    (1.20) |
| | | II | 1.77    (1.30) |
| | | IIs | 1.41    (1.20) |
| B | Right bank closure dyke | I | covered by FoS in II |
| | | Is | covered by FoS in IIs |
| | | II | 1.92    (1.30) |
| | | IIs | 1.57    (1.20) |
| | | V | covered by FoS in IIs |
| | | Vs | 1.45    (1.20) |
| | | VI | 2.14    (1.30) |
| C | Diversion canal closure    upstream slope: | V | covered by A-I |
| | | Vs | covered by A-Is |
| | downstream slope: | V | covered by FoS in VII |
| | | Vs | 1.61    (1.20) |
| | | VI | 1.93    (1.30) |
| D | Construction pit | III | 1.54    (1.30) |
| | | IIIs | 1.22    (1.20) |
| | | IV | 1.31    (1.20) |

# A 7.1-1: Slope Stability Analysis, Diversion Canal, Sheet 1

A: Diversion Canal Slope

Load Combination I

1.812

20 kN/m

Holocene Silt

Pleistocene Sand & Gravel

Water (57.90)
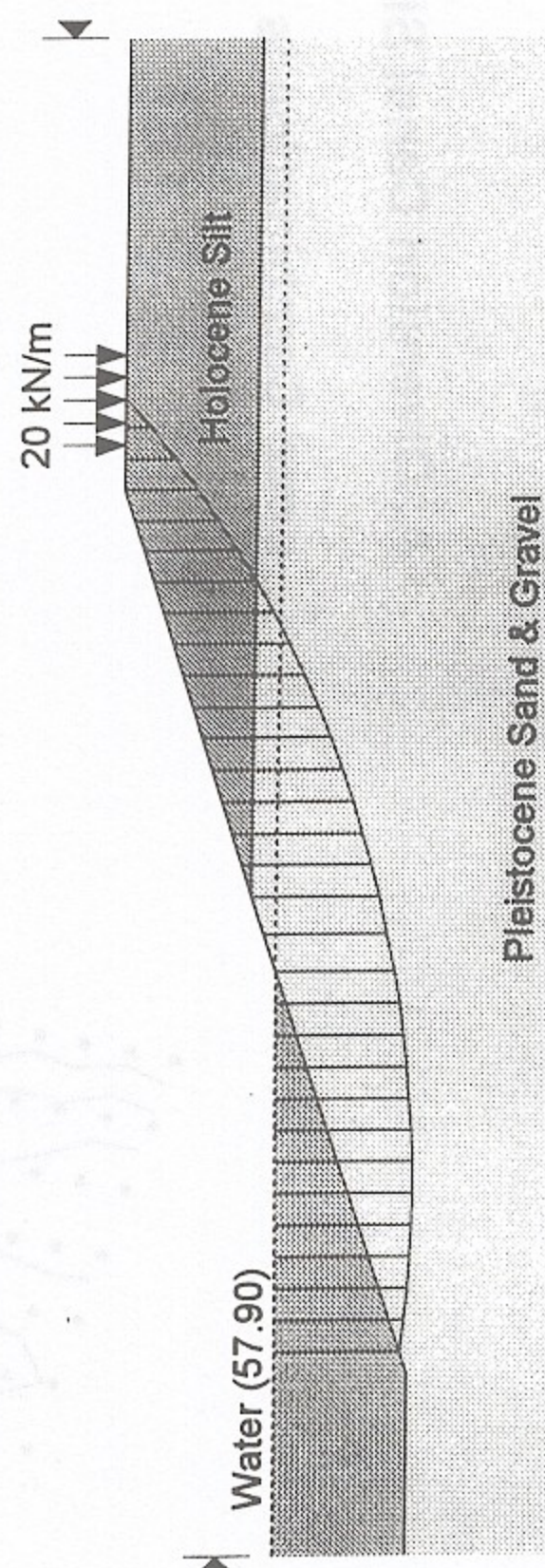
# A 7.1-1: Slope Stability Analysis, Diversion Canal, Sheet 2

A: Diversion Canal Slope

Load Combination Is

1.413

20 kN/m

Holocene Silt

Pleistocene Sand & Gravel

Water (57.90)

File Name: diversion1.slp
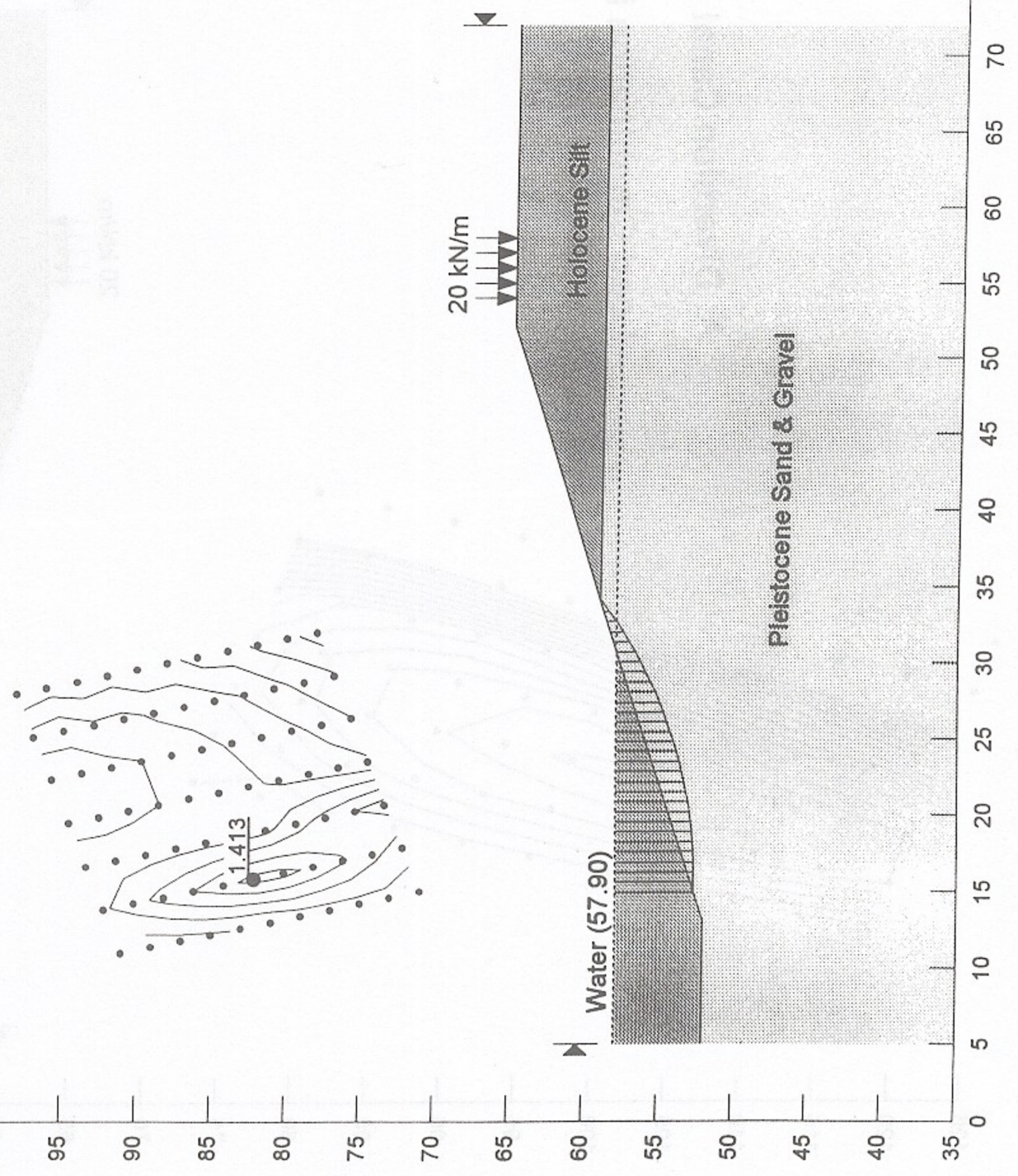
# A 7.1-1: Slope Stability Analysis, Diversion Canal, Sheet 3

A: Diversion Canal Slope

Load Combination II



1.768

40 kN/m

Holocene Silt

Pleistocene Sand & Gravel

Water (57.90)
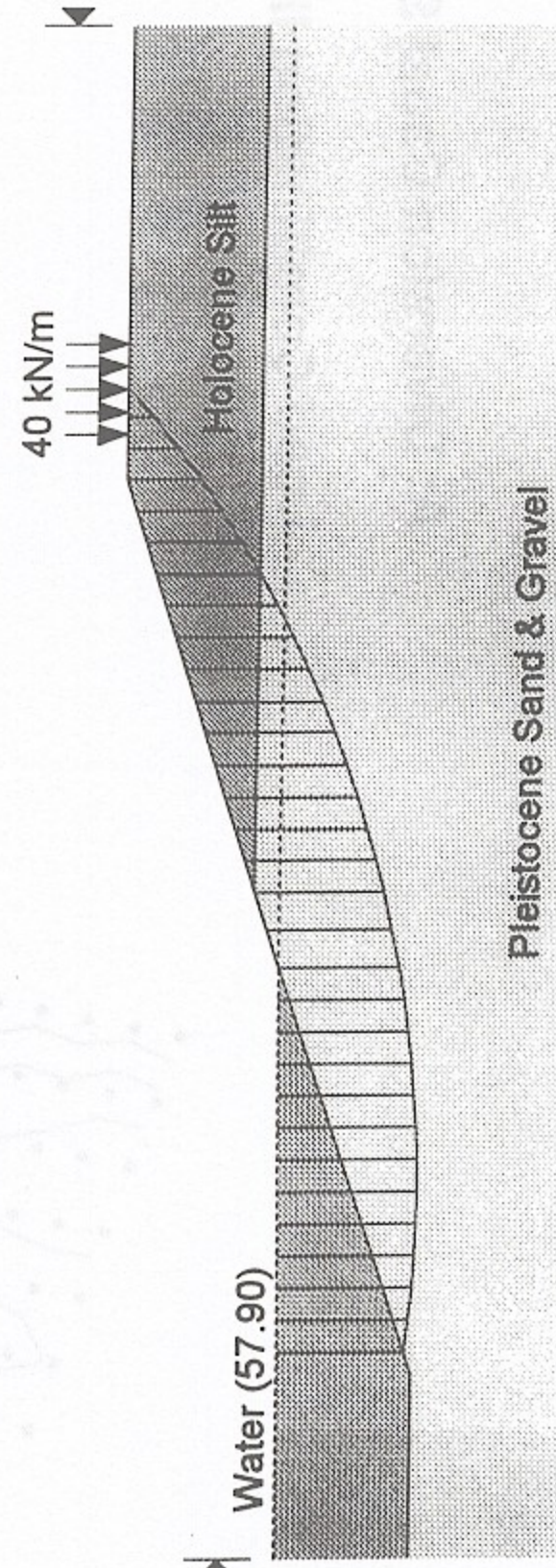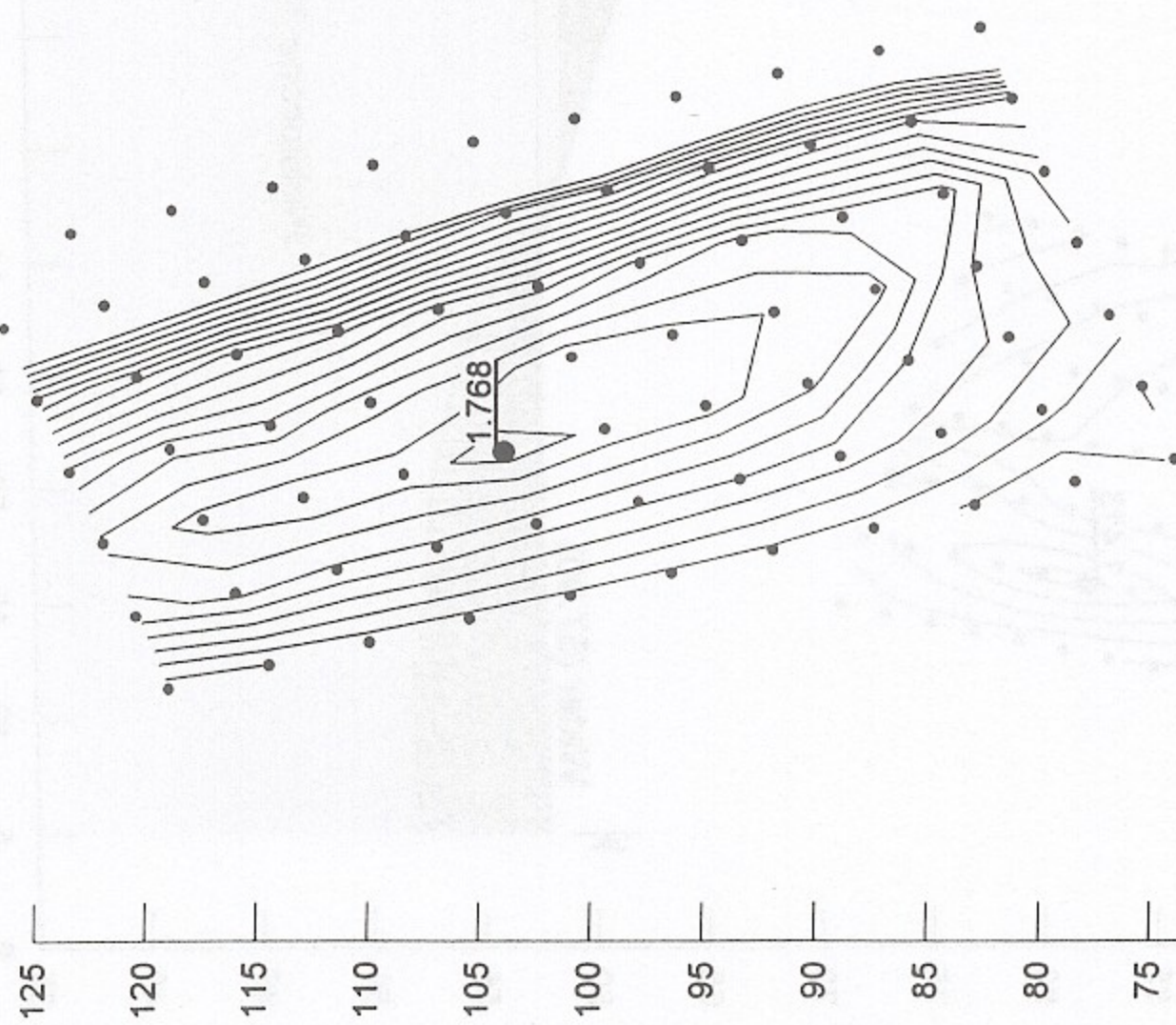
# A 7.1-1: Slope Stability Analysis, Diversion Canal, Sheet 4

A: Diversion Canal Slope

Load Combination IIs

1.413

40 kN/m

Holocene Silt

Pleistocene Sand & Gravel

Water (57.90)
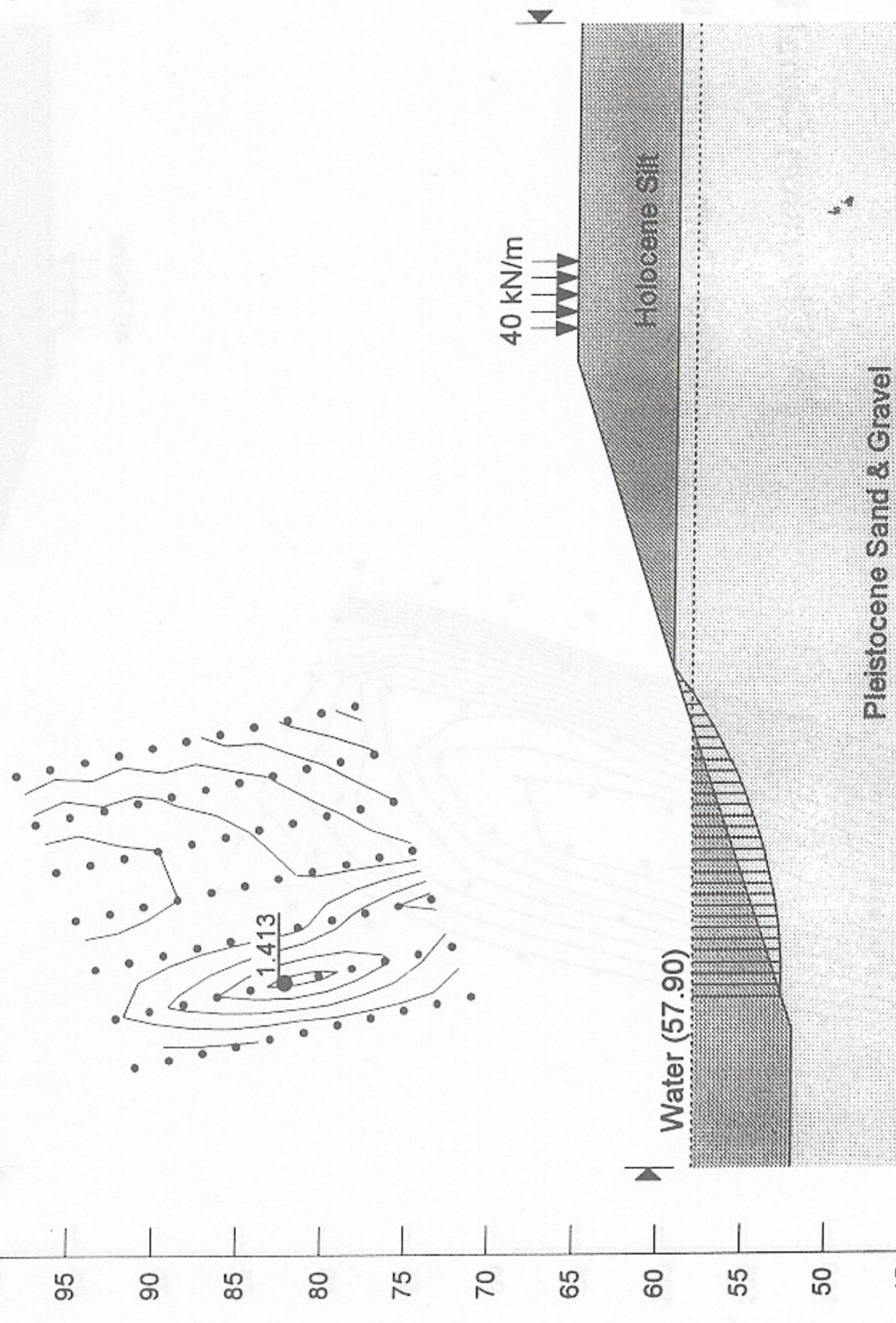
A 7.1-2: Slope Stability Analysis, Right Bank Closure Dyke, Sheet 1

B: Right Bank Closure Dyke

Load Combination II

40 kN/m2

Silty Fill

Sand & Gravel

Pleistocene Sand & Gravel (in-situ)

Water (62.5)

1.919
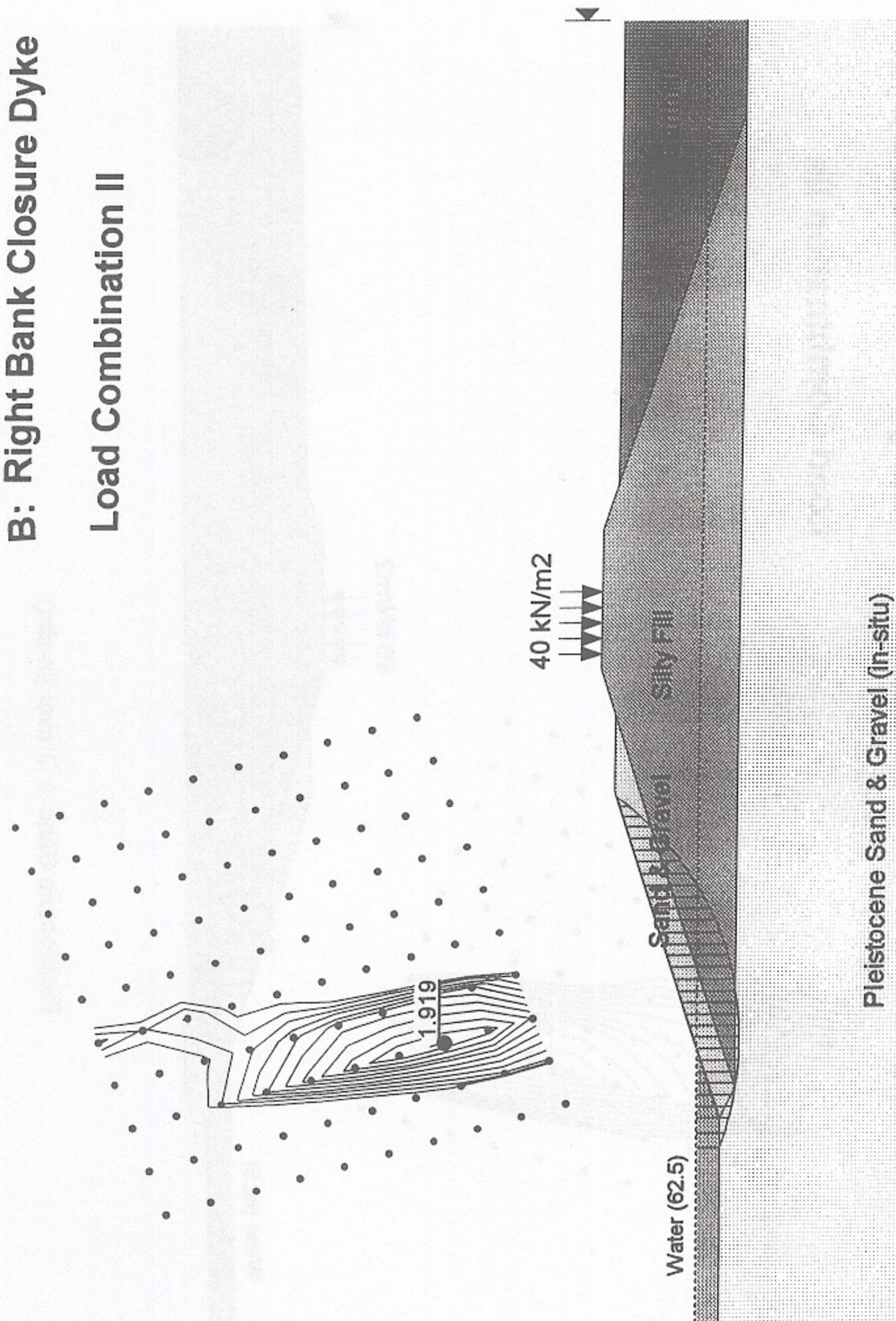
File Name: dyke1.slp

A 7.1-2: Slope Stability Analysis, Right Bank Closure Dyke, Sheet 2

B: Right Bank Closure Dyke

Load Combination IIs

40 kN/m2

Silty Fill

Sand & Gravel

Pleistocene Sand & Gravel (In-situ)

Water (62.5)

1.569
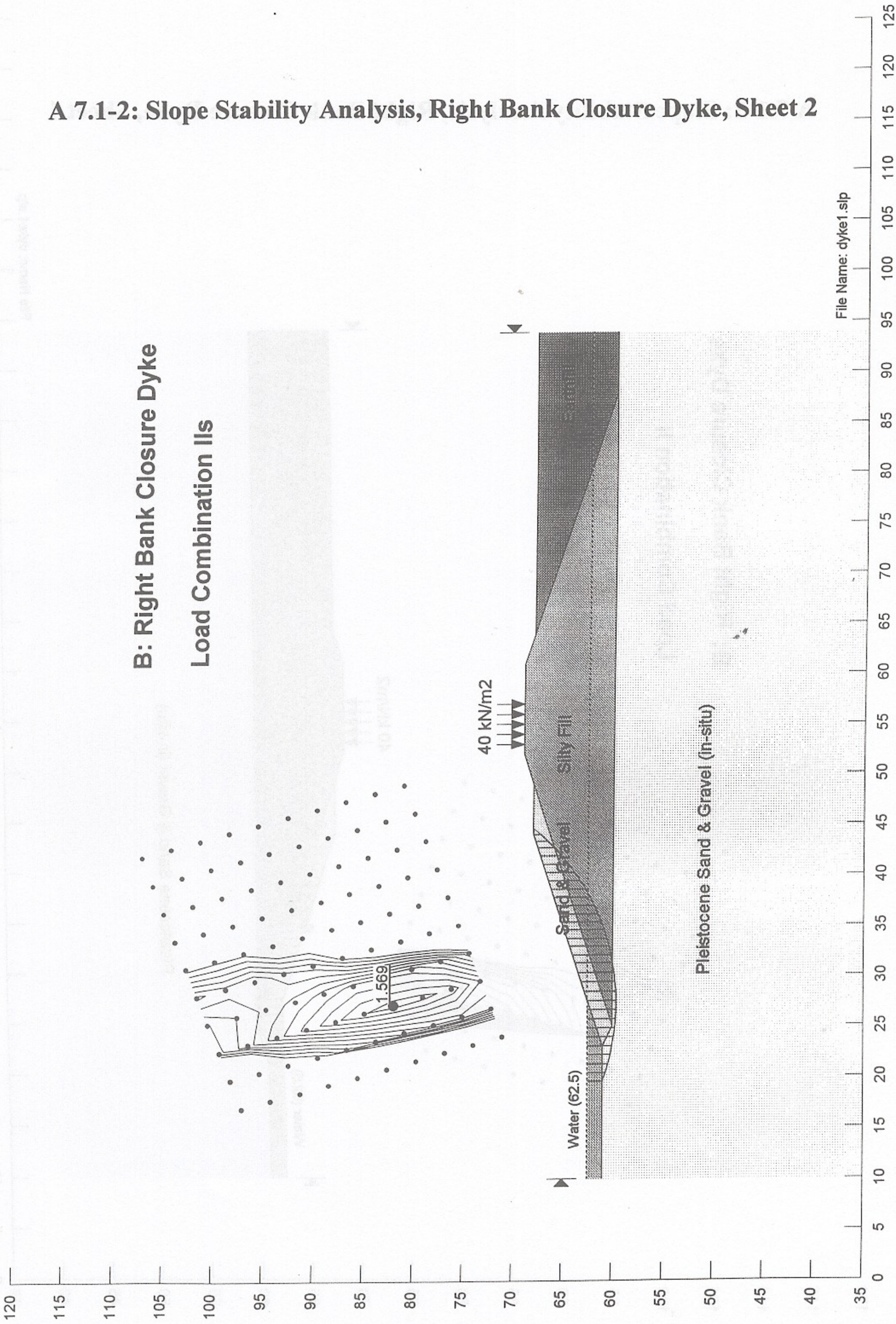
File Name: dyke1.slp

# A 7.1-2: Slope Stability Analysis, Right Bank Closure Dyke, Sheet 3

**B: Right Bank Closure Dyke**

**Load Combination Vs**

1.447

20 kN/m2

Sand & Gravel

Silty Fill

Water (65.9)
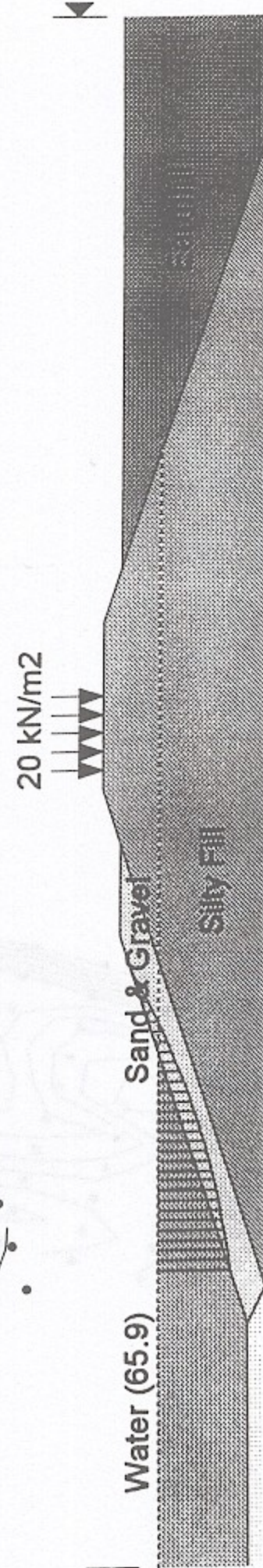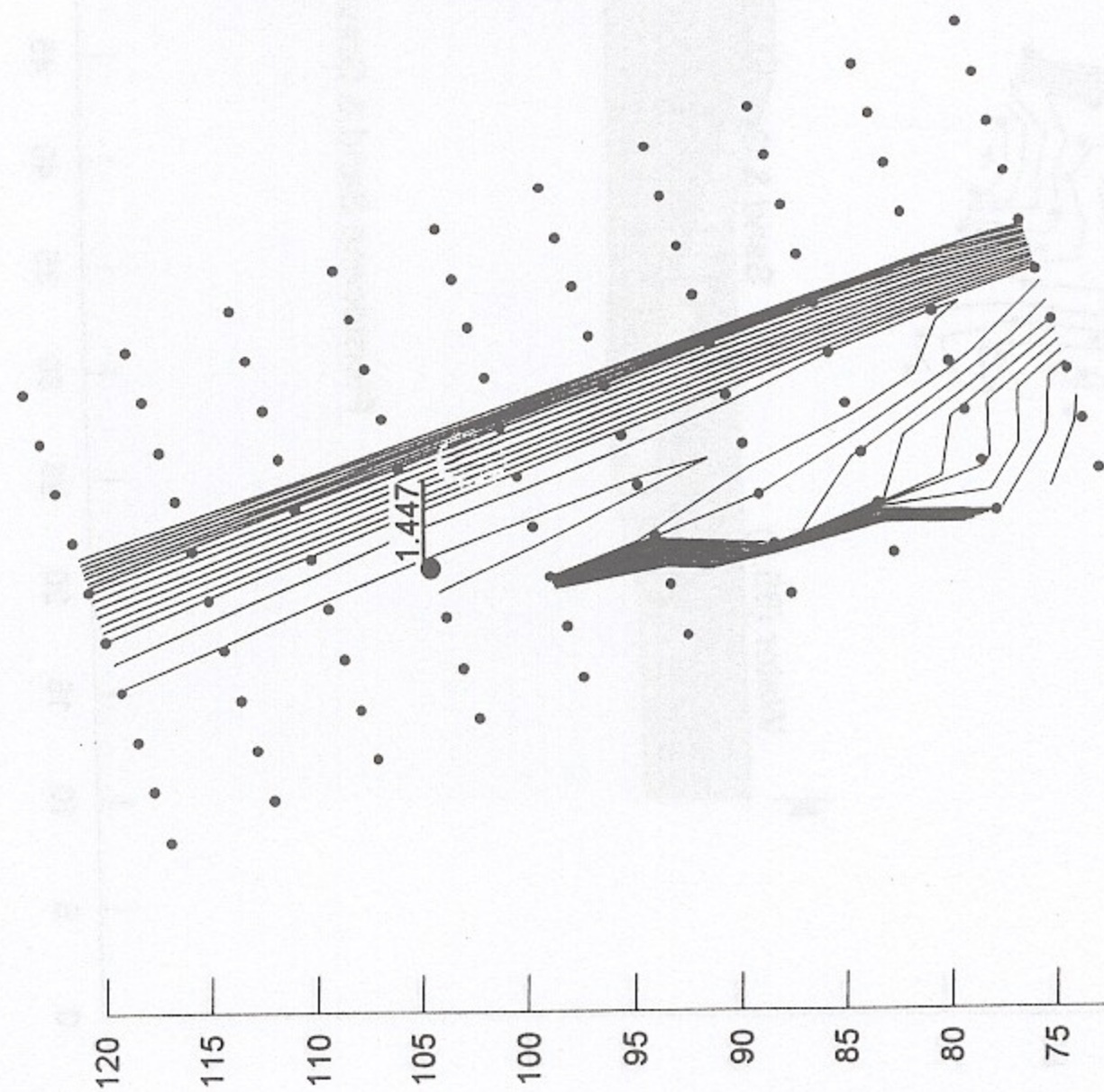
# A 7.1-2: Slope Stability Analysis, Right Bank Closure Dyke, Sheet 4

## B: Right Bank Closure Dyke

## Load Combination VI



40 kN/m2

Silty Fill

Sand & Gravel

2.140

Water (65.9)
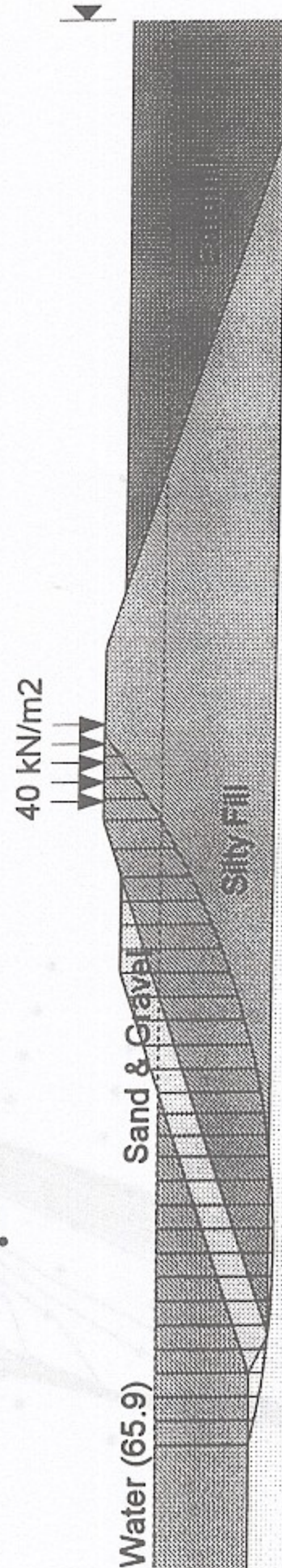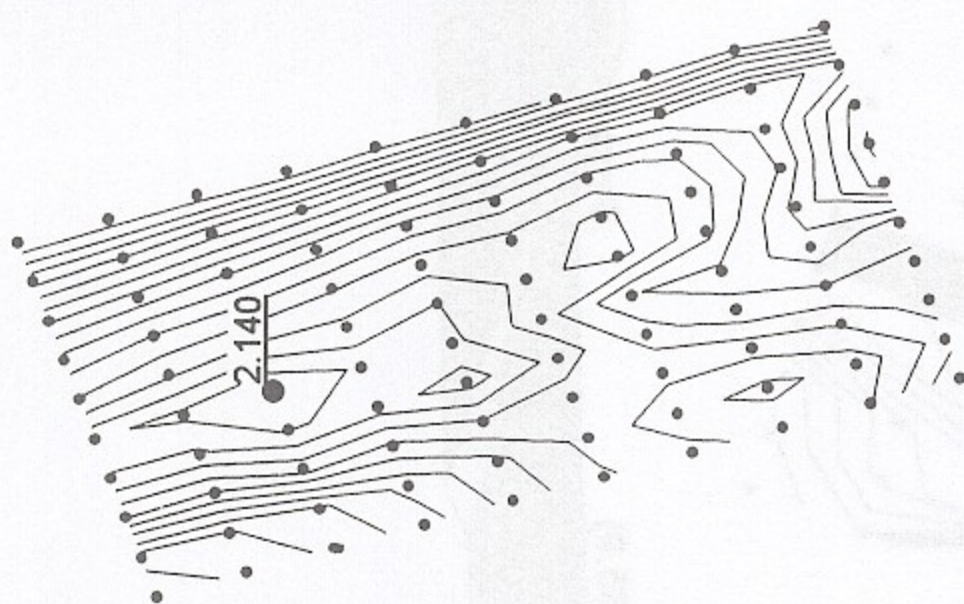
Pleistocene Sand & Gravel (in-situ)

File Name: dyke2.slp

C: Diversion Canal Slope
Downstream Slope of Closure Dam

Loadcase Combination Vs

1.612

20 kN/m2

Water (62)

Compacted Sand & Gravel Fill

Pleistocene Sand & Gravel (in-situ)

File Name: closure2.slp
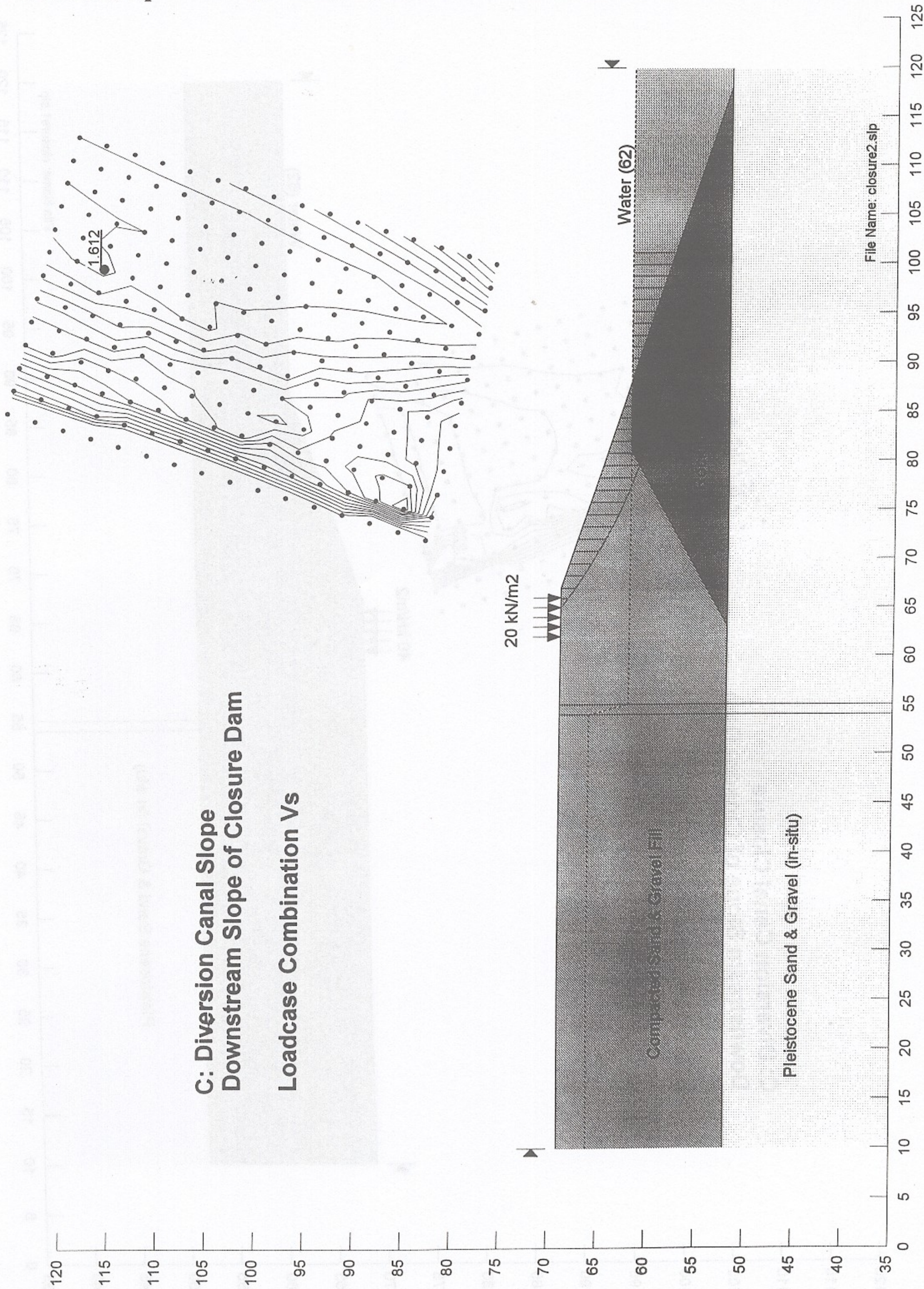
C: Diversion Canal Closure
Downstream Slope of Closure Dam

Loadcase Combination VI

Water (62)

40 kN/m2

1.928

Pleistocene Sand & Gravel Fill

Pleistocene Sand & Gravel (in situ)

File Name: closure1.slp
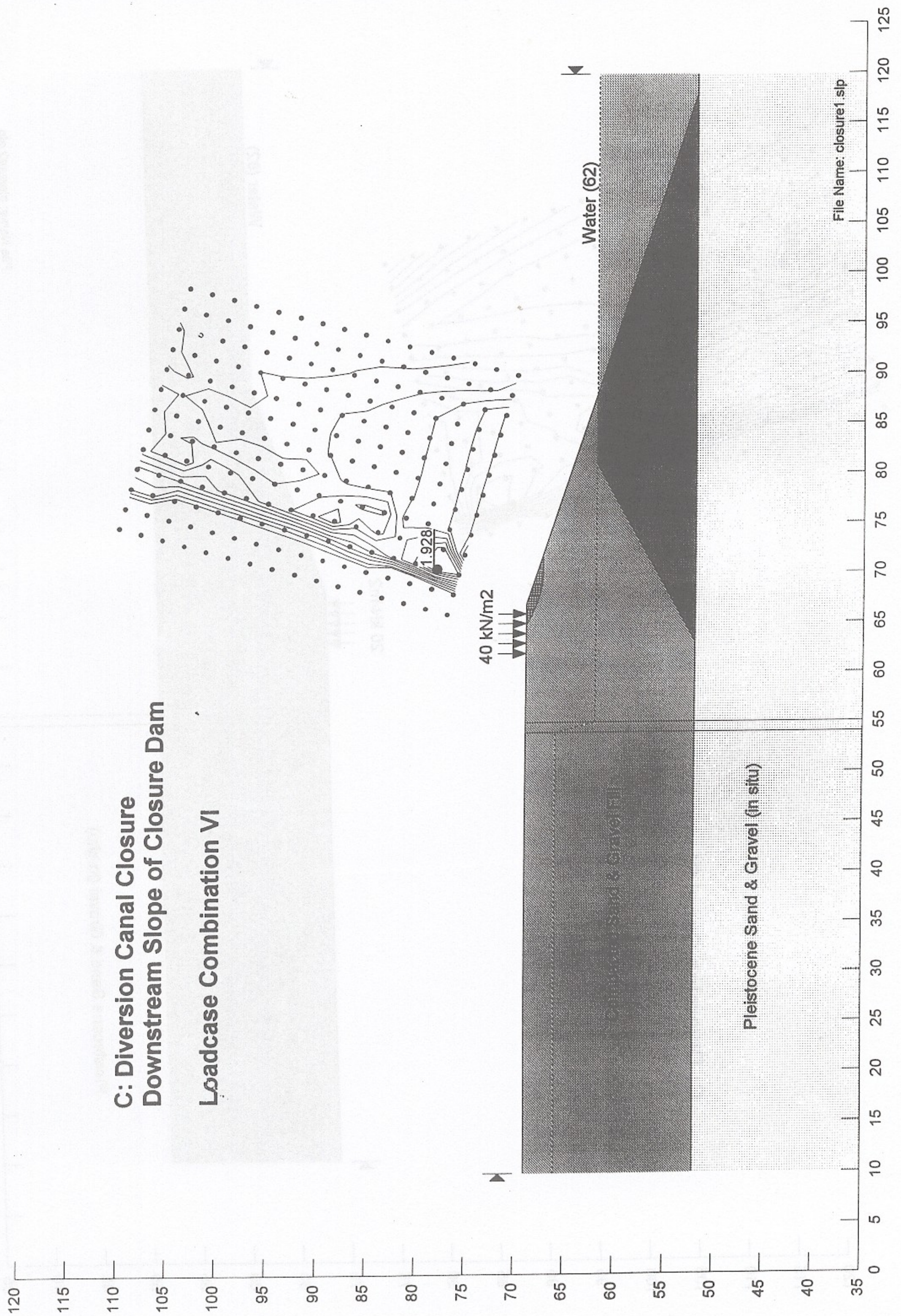
# A 7.1-4: Slope Stability Analysis, Construction Pit, Sheet 1

D: Construction Pit Left Bank Slope

**Load Combination III**

Case 1: critical high waterlevel
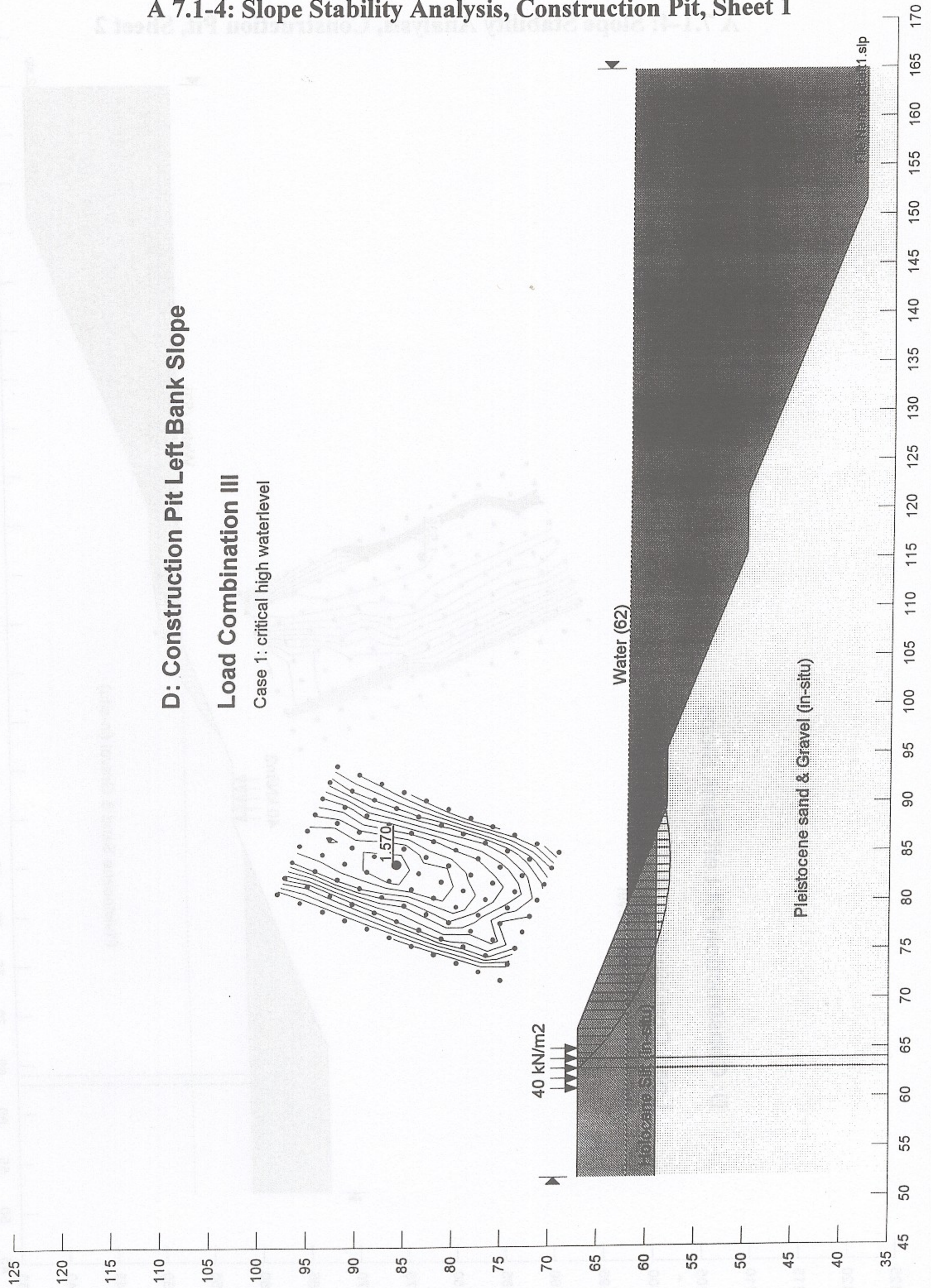


Water (62)

Pleistocene sand & Gravel (in-situ)

Holocene Silt (in-situ)

40 kN/m2

1.570

Bustlug_sheet1.slp

# D: Construction Pit Left Bank Slope

## Load Combination III

Case 2: critical low waterlevel

Water (53)

Holocene Silt (in-situ)

Pleistocene Sand & Gravel (in-situ)

40 kN/m2

1.539

FleNamesheet2.slp

# A 7.1-4: Slope Stability Analysis, Construction Pit, Sheet 3



**D: Construction Pit Left Bank Slope**

**Load Combination IIIs**

Case 1: critical high waterlevel

Water (62)

40 kN/m2

Holocene Silt (in-situ)

Pleistocene sand & Gravel (in-situ)

1.320

Pleistocene sheet1.slp
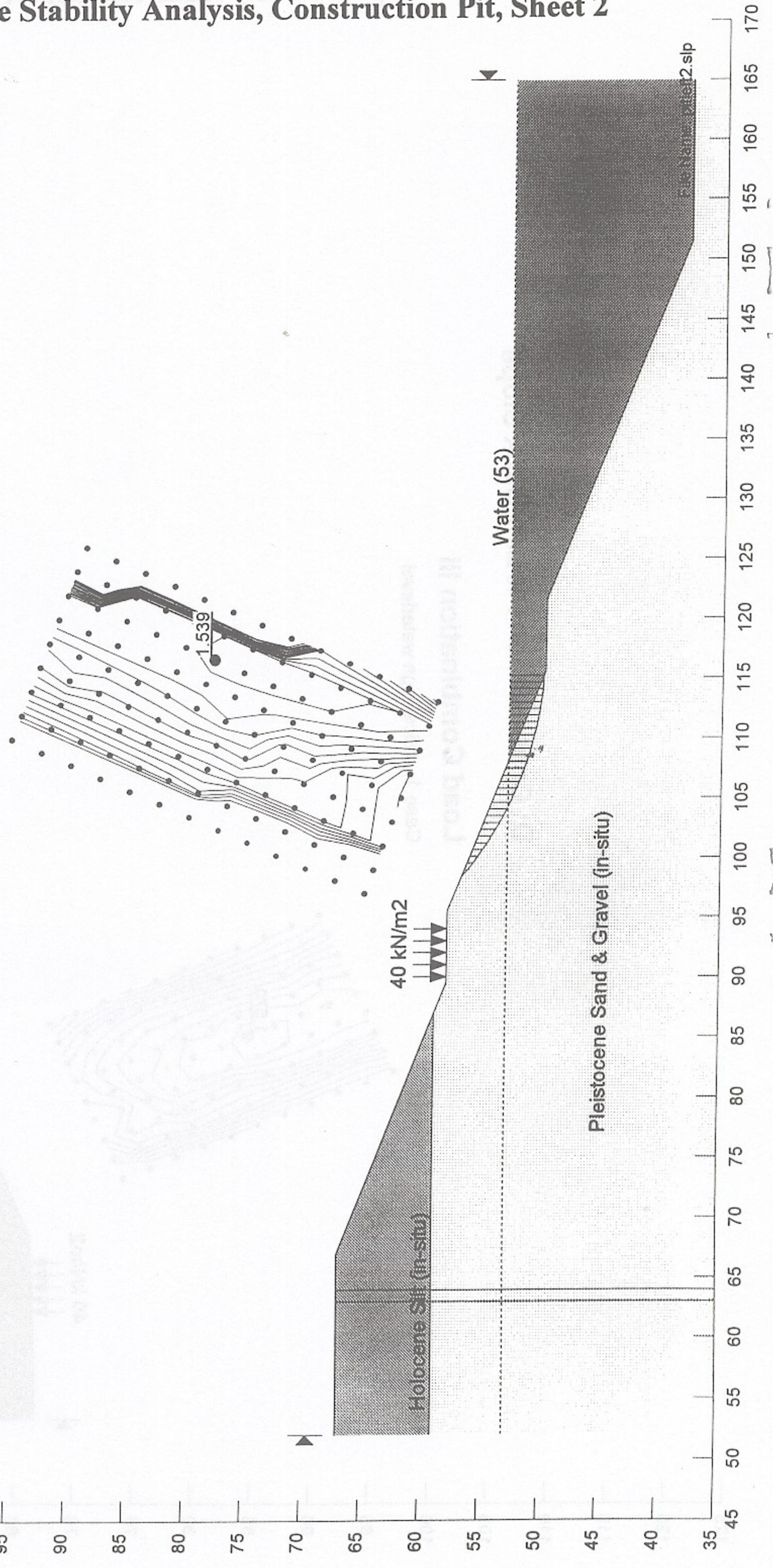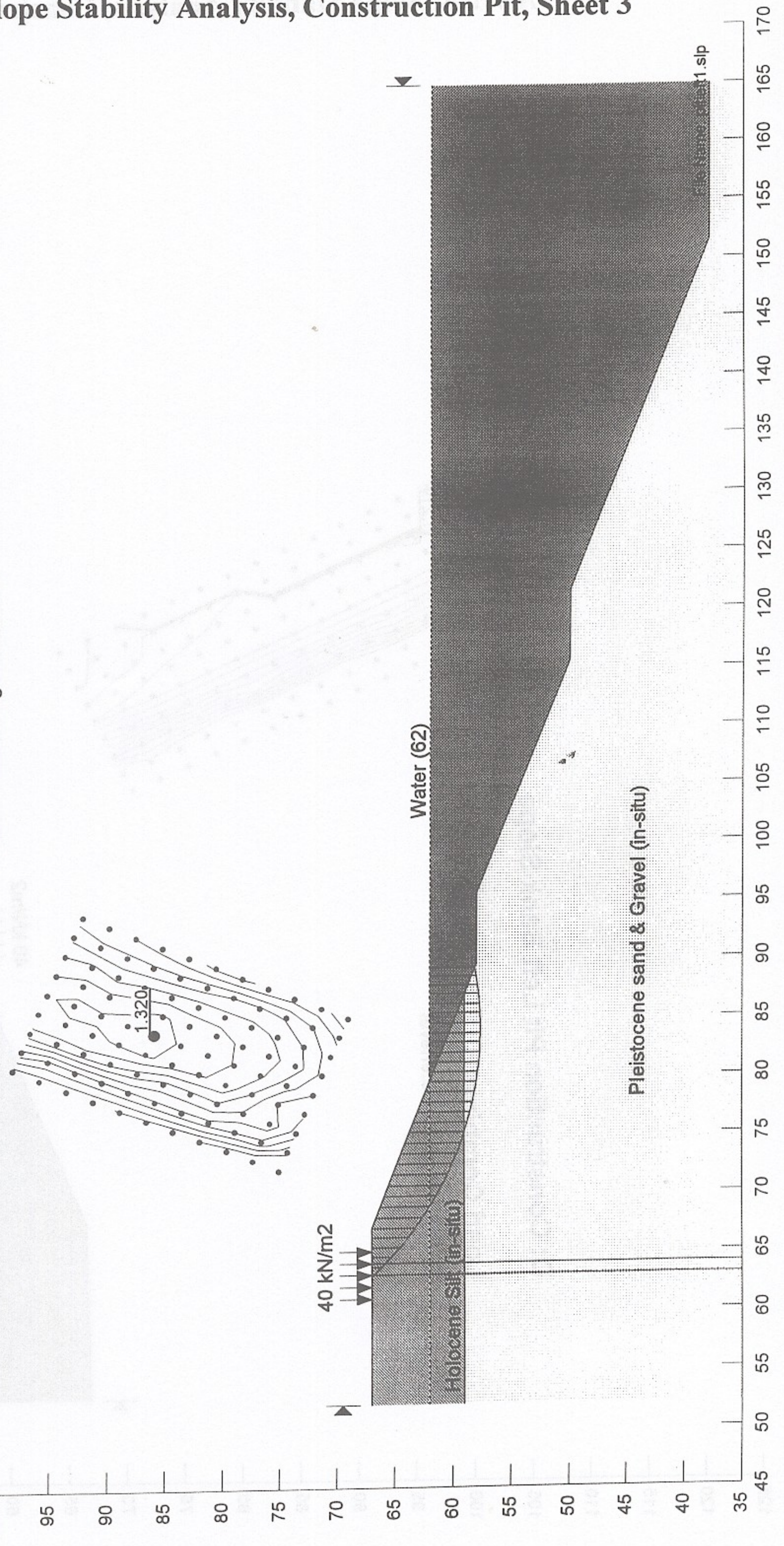
# A 7.1-4: Slope Stability Analysis, Construction Pit, Sheet 4

## D: Construction Pit Left Bank Slope

## Load Combination IIIs

Case 2: critical low waterlevel

1.219

40 kN/m2

Water (55)

Holocene Silt (In-situ)

Pleistocene Sand & Gravel (In-situ)
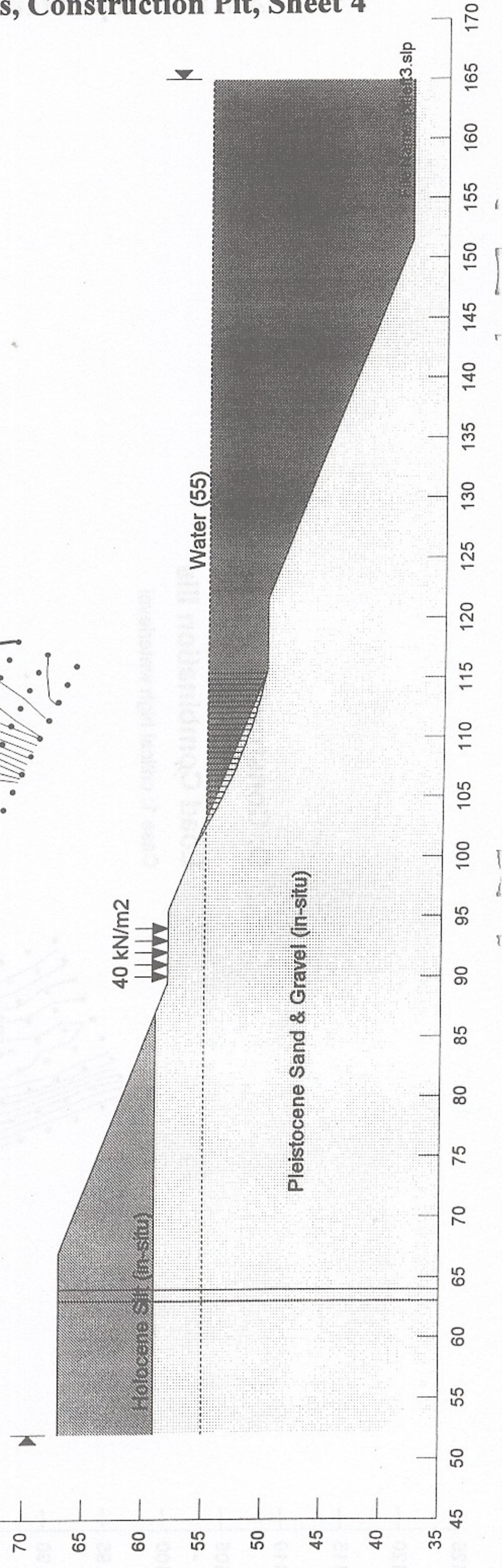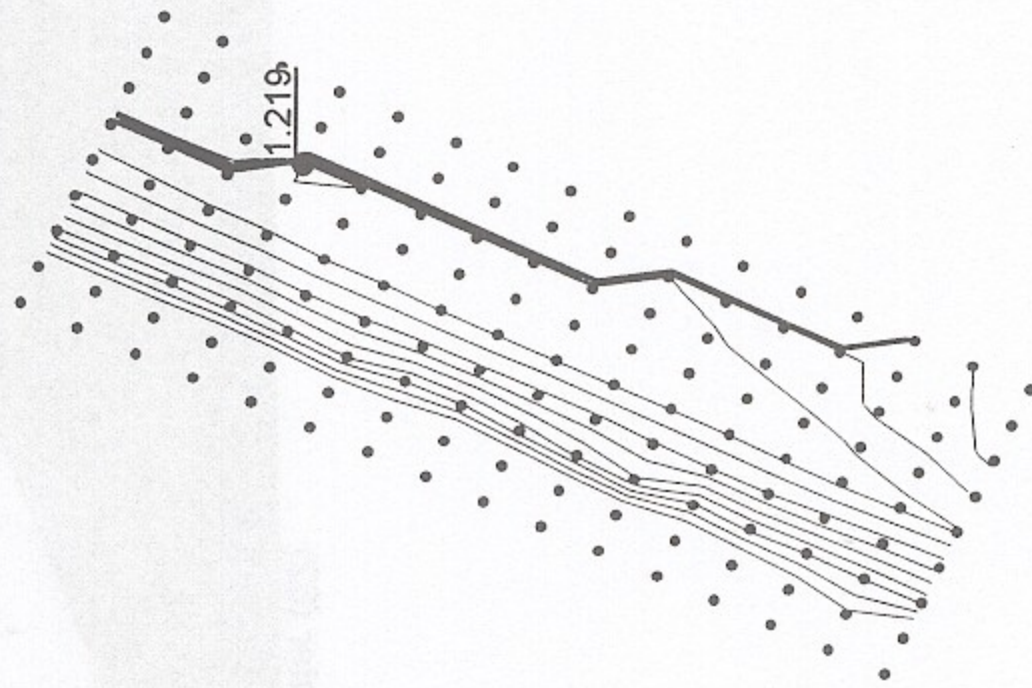
# A 7.1-4: Slope Stability Analysis, Construction Pit, Sheet 5

D: Construction Pit Left Bank Slope

Load Combination IV

Water (50)

1.305

40 kN/m2

Pleistocene Sand & Gravel (in-situ)

Holocene Silt (in-situ)

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# Appendix 7-2

# Seepage Analyses under and around Permanent Structures

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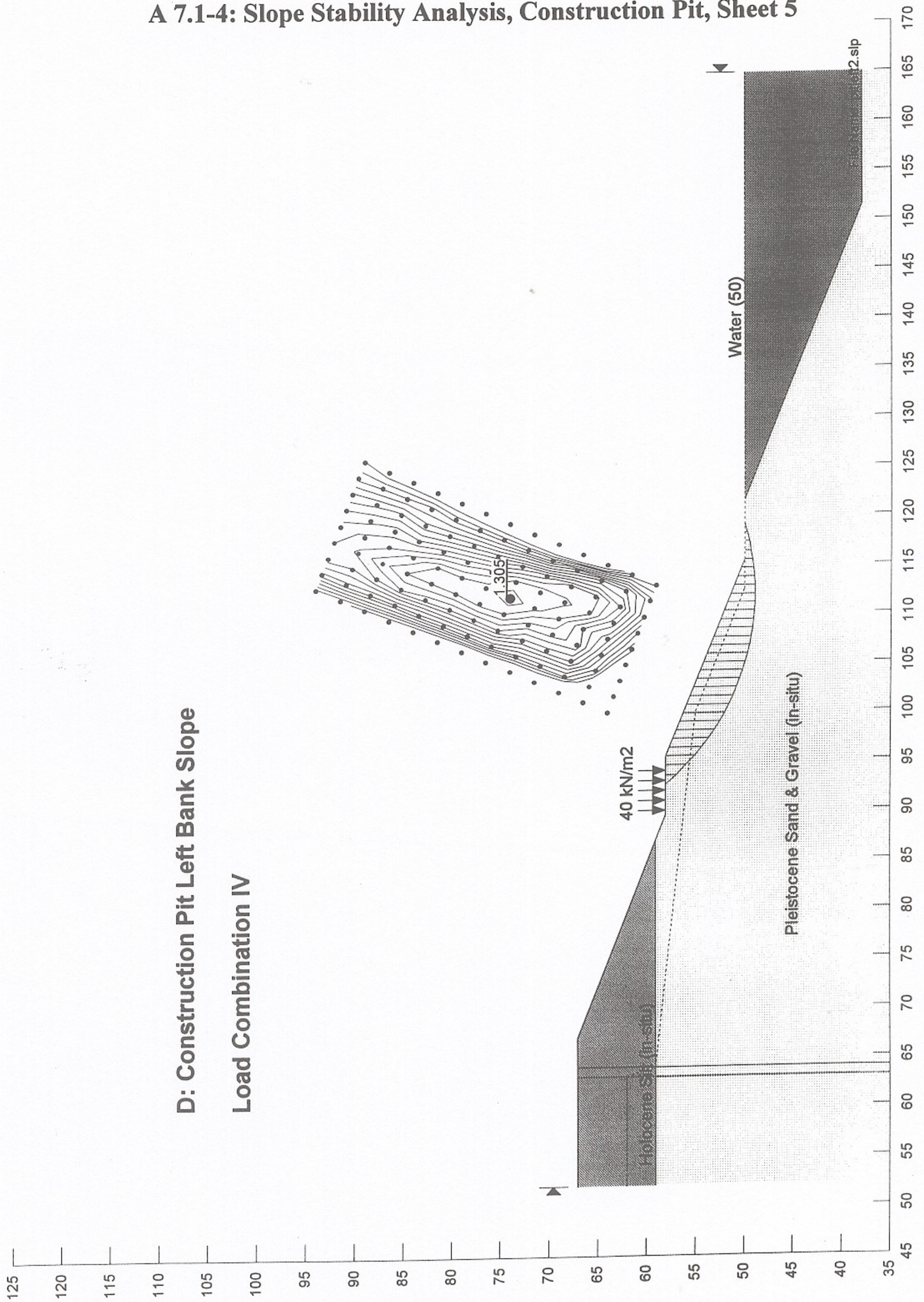
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## A 7-2.1 General

For the Tender Design of the New Naga Hammadi Barrage, various calculations had to be carried out for the following purposes:

- Optimisation of the depth of the permanent cut-off wall under the structures (powerhouse, sluiceway, navigation locks).
- Optimisation of the depth and length of the closure dam cut-off wall (left bank) and the right bank cut-off wall (if necessary).
- Determination of seepage and uplift forces under the structures.
- Calculation of exit gradients and flow velocities at the downstream toe of the closure embankment.

## A 7-2.2 Methodology of Model Application.

The reliability of calculated uplift forces and gradients depend on the accuracy of the assumed seepage flow pattern. With respect to the geological conditions and the project layout, various vertical and horizontal groundwater flow components will occur. The three-dimensional effects of restrictions in aquifer flow resulting from the construction can only be represented by a full three-dimensional finite element model. On one hand, a 3D-flow-model permits a realistic spatial discretisation of the complex geometry of the structure, especially the positions and design of planned diaphragm walls. On the other hand only full 3D-models are able to calculate the resulting 3D-flowfield in a realistic and reliable way.

The general method of groundwater model application consists of the following steps:

- Definition of the model area.
- Definition of the 3D-discretisation.
- Definition of geologic conditions.
- Definition of boundary (hydrologic) conditions.
- Simulation of the system under proposed conditions.
- Documentation and graphical presentation of the data inventory, model development and simulation results.

The modelling software used for the above mentioned purpose is a full three dimensional steady state groundwater model (GW3DS). Its essential attributes are numerical integration with the finite element method, spatial discretisation by tedrahedric elements and 3D-flow with free groundwater surface.

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## A 7-2.3 Definition of the Model Area

The size of the model area has to be such, that changes of the flowfield within the model area, which will occur during the simulation runs, do not effect the flow conditions at the boundaries. For this, the model area extends over a larger size then the project area itself. The project area is shown in **Figure A 7-2.1**. The upstream boundary is represented by the location of the existing Naga Hammadi Barrage. The downstream boundary is defined some 3 km downstream of the New Barrage. The left and right bank boundaries are located in an equal distance to the corresponding Nile River bank.

The complete model area is reflected by the finite element net covering an area of approx. 41 km².

## A 7-2.4 Geological Conditions

The geology defined in the groundwater model must accurately reflect the observed geologic structure of the region. Main information about the local stratigraphic succession in the vicinity of the New Barrage is given by the interpretation of the geotechnical investigations, **Volume 4, Table 3.1**.

## A 7-2.5 Discretisation

The spatial discretisation of the 3D-flow-model was carried out in close adaptation to the existing geologic and morphologic conditions of the site as well as to the structural and hydrogeologic details of the project. The FE-discretisation consists of nodes (representing real x, y and z-coordinates) and tedrahedron elements. The mesh was refined in areas which are assumed to be affected by complex flow conditions (**Figure A 7-2.2**), while larger elements are used for peripheral regions of the model area (**Figure A 7-2.3**).

The horizontal discretisation reflects topographic elements like the River Nile and the diversion canal as well as artificial structures like the temporary and permanent cut-off walls, sheet piles, and the layout of the structures (powerhouse, sluiceway and navigation locks). Cut-off walls are represented by sections of elements with an average length of 50 m and the given thickness of 0.8 m.

The horizontal discretisation of the model area is characterised as follows:

- Number of nodes: 1,793 (per layer)
- Number of elements: 3,534 (per layer)
- Area: 41 km²
- Minimum element size: 2 m²
- Average element size: 11,655 m²
- Maximum element size: 319,254 m²
- Aerial ratio: 1:159,595

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The definition of the vertical discretisation is determined by the strata described in **Section A 7-2.6**. In total, nine 3D-layers were defined to represent the upper and lower sand and the intercalating gravel and clay units (see **Table A 7-2.1**). The aquifer base for the model is given by the Protonile-Prenile deposits, which are interpreted as aquicludes for the overlying aquifer system. Its depth at the location of the New Barrage is within a range of some 120 m below ground (determined by the analysis of Vertical Electrical Soundings).

As each 3D-element is represented by three tedrahedrons, the full 3D-discretisation results in 17,930 3D-nodes and 95,418 tedrahedrons.

### A 7-2.6 Material Properties of Geologic Strata and Structures

The geologic and hydrogeologic parameters of the porous aquifer are derived under consideration of permeability, $k_h/k_v$ relations and stratigraphic inhomogenities, as known so far. The calculations were performed for two ratios of horizontal to vertical permeabilities, of 10/1 and 50/1. This variation was carried out in order to cover a wider range of local changes in the stratification due to the significant influence of that ratio on the exit gradients.

The following permeabilities and soil stratifications were used in accordance with the evaluation of the geotechnical field investigations of earlier studies.

#### Table A 7-2.1: General Vertical Structure of Model Layout

| 3D-Layer | Depth from m asl | Depth to m asl | Strata | Horizontal Permeability m/s | Horizontal/ Vertical Permeability |
|----------|------------------|----------------|--------|-----------------------------|------------------------------------|
| 1 | 67.50 | 53.00 | Sand | $1 \cdot 10^{-3}$ | 10 and 50 |
| 2 | 53.00 | 46.00 | Sand | $1 \cdot 10^{-3}$ | 10 and 50 |
| 3 | 46.00 | 41.00 | Sand | $1 \cdot 10^{-3}$ | 10 and 50 |
| 4 | 41.00 | 39.00 | Sand & Gravel | $2 \cdot 10^{-3}$ | 1 |
| 5 | 39.00 | 27.00 | Sand | $1 \cdot 10^{-3}$ | 10 and 50 |
| 6 | 27.00 | 15.00 | Sand | $1 \cdot 10^{-3}$ | 10 and 50 |
| 7 | 15.00 | 2.00 | Clay | $1 \cdot 10^{-7}$ | 10 and 50 |
| 8 | 10.00 | -10.00 | Sand | $1 \cdot 10^{-4}$ | 10 and 50 |
| 9 | -10.00 | -50.00 | Sand | $1 \cdot 10^{-4}$ | 10 and 50 |

In addition to the material types representing the geological structure, artificial structures as cut-off walls, sandfill, rockfill or riprap are defined as listed in **Table A 7-2.2**.

## Table A 7-2.2: Material Properties of Structures and Sandfill

| Material Type | Horizontal Permeability m/s | Horizontal/ Vertical Permeability |
|---|---|---|
| Cut-off Walls | $1 \cdot 10^{-8}$ | 1 |
| Sandfill | $1 \cdot 10^{-3}$ | 1 |
| Rockfill | $1 \cdot 10^{-2}$ | 1 |
| Riprap | $5 \cdot 10^{-2}$ | 1 |

## A 7-2.7 Boundary Conditions

Boundary conditions were defined to describe the groundwater inflow and outflow across the model boundaries. For the upstream and downstream boundaries groundwater levels are given as DIRICHLET-type prescribing the piezometric head at these locations (66.00 m for the upstream boundary and 57.00 m for the downstream boundary).

Nile water levels were defined as constant head for all nodes representing the riverbed in the upper layer. The headpond is given with 65.90 m asl, while tailwater levels are about 57.06 m asl, representing the minimum water level at a river discharge of 350 m³/s and under consideration of degradation of the downstream riverbed.

The eastern and western boundaries are regarded as no-flow boundaries.

## A 7-2.8 Simulation Results

### A 7-2.8.1   Layout of Cut-off Walls

The permanent cut-off wall below the structures is connected to the temporary cut-off wall of the construction pit at both river banks. Along the powerhouse, sluiceway and navigation locks the cut-off wall penetrates into the clay layer.

As the clay layer shows a significant slope from east to west, the depths of the cut-off wall is some 45 m below surface besides the navigation locks and some 55 m below surface besides the powerhouse.

A lateral extension beyond the diaphragm of the construction pit at the right bank is not necessary. Various calculations (**Appendix 7-2**) have shown, that the increment of the flowpath by an extent of the cut-off wall into the right bank (some 100 m to 200 m) is to small compared to the existing flowpath of some 600 m. As a result, a significant improvement concerning the right bank exit gradients was not assessed.

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The optimisation of the cutoff wall (**Appendix 7-2**) layout results in a 330 m long intersection through the closure embankment and powerhouse backfill as shown on **Album No. 70**. The temporary cut-off wall of the construction pit completely intersects the aquifer and keys into the clay layer. The top level of this wall is given with 66.00 m asl.

### A 7-2.8.2   Waterlevels

The distribution of heads resulting from the 3D-model calculation for the minimum tailwater level of 57.06 m asl (for which the exit gradients are at maximum) is shown in **Figure A 7-2.4** for the upper layer (surface layer). In general the groundwater flows around the temporary cut-off wall of the construction pit (**Figure A 7-2.4**).

The permanent cut-off wall completely intersects seepage below the structures. Significant seepage only occurs between the powerhouse abutment pier and the left bank connection of the temporary diaphragm, where the permanent diaphragm wall does not penetrate into the clay layer. The downstream section of the powerhouse abutment pier needs protection by rockfill in order to avoid critical exit gradients.

The left bank cut-off wall is bypassed and understreamed. The incremental length of the flowpath results in a significant reduction of piezometric heads behind the cut-off wall. The groundwater level reduces to about 59.00 m asl behind the wall.

Cross Sections under the closure embankment, powerhouse, sluiceway and navigation lock show the vertical flowfield under these structures (**Figures A 7-2.5 to A 7-2.8**).

### A 7-2.8.3   Flow Velocities

Flow velocities (DARCY velocities $v_f$) usually vary within a range of $1 \cdot 10^{-5}$ to $6 \cdot 10^{-5}$ m/s. Maximum velocities within a range of $1 \cdot 10^{-4}$ to $5 \cdot 10^{-4}$ m/s occur at locations with groundwater outflow into the rockfill at the closure embankment and the powerhouse abutment pier.

### A 7-2.8.4   Exit Gradients

Exit gradients were evaluated for both ratios of horizontal to vertical permeability for the closure embankment, the powerhouse abutment pier and the right bank groundwater outflow. The maximum exit gradients and the average exit gradients for these locations were computed as follows:

**(1)     Closure Dam**

- horizontal/vertical permeability 10/1:     $I_{max} = 0.04$     $I_{average\ e} = 0.02$
- horizontal/vertical permeability 50/1:     $I_{max} = 0.02$     $I_{average} = 0.01$

**(2)     Powerhouse Abutment Pier**

- horizontal/vertical permeability 10/1:     $I_{max} = 0.03$     $I_{average} = 0.02$
- horizontal/vertical permeability 50/1:     $I_{max} = 0.02$     $I_{average} = 0.01$

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### (3)    Right Bank Outflow

- horizontal/vertical permeability 10/1:    $I_{max} = 0.06$    $I_{average} = 0.04$
- horizontal/vertical permeability 50/1:    $I_{max} = 0.04$    $I_{average} = 0.03$

In all cases, the exit gradients are far less than the "critical" gradient of about 0.2 for fine and medium sand. The influence of the ratio on the exit gradients is obvious, as higher ratios lead to smaller gradients (**Figure A 7-2.9**).

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Tallwater

57.05

Diversion
Canal

Power-
house

Sluiceway

Closure Dam
Cutoff Wall

Rock-
fill

Closure Dam

Permanent
Cutoff Wall

Diversion
Canal
Backfill

Abutment-
Pier

Headpond

65.90

Navigation
Locks

Temporary
Cutoff Wall

RIVER
NILE

```
0    100    200    300    400
              m
```

**A 7-2.1: Location Plan of Structures**

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## A 7-2.2: Horizontal Discretization

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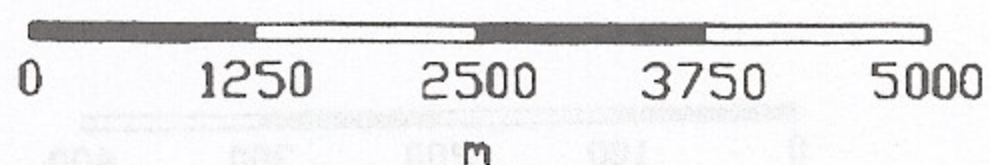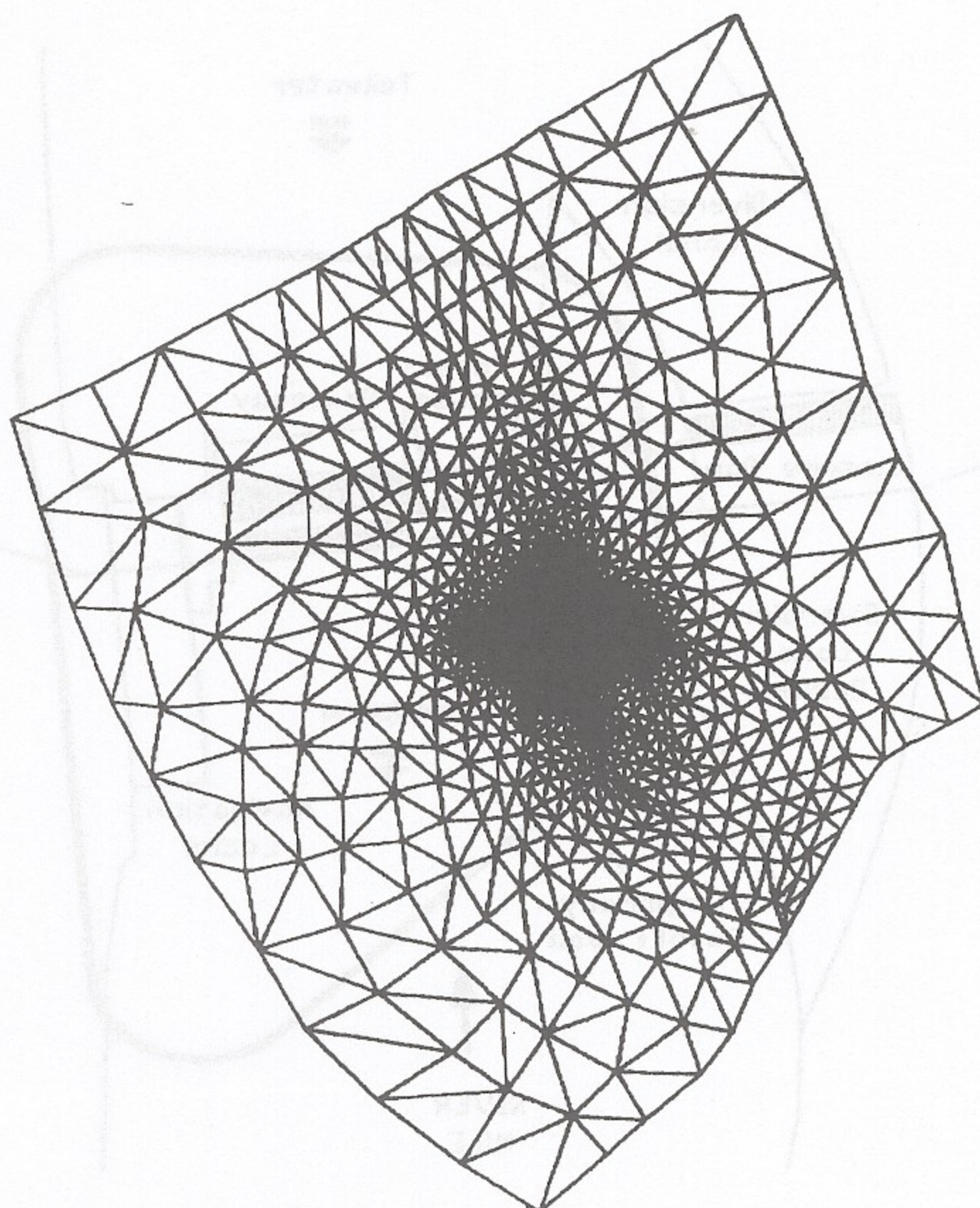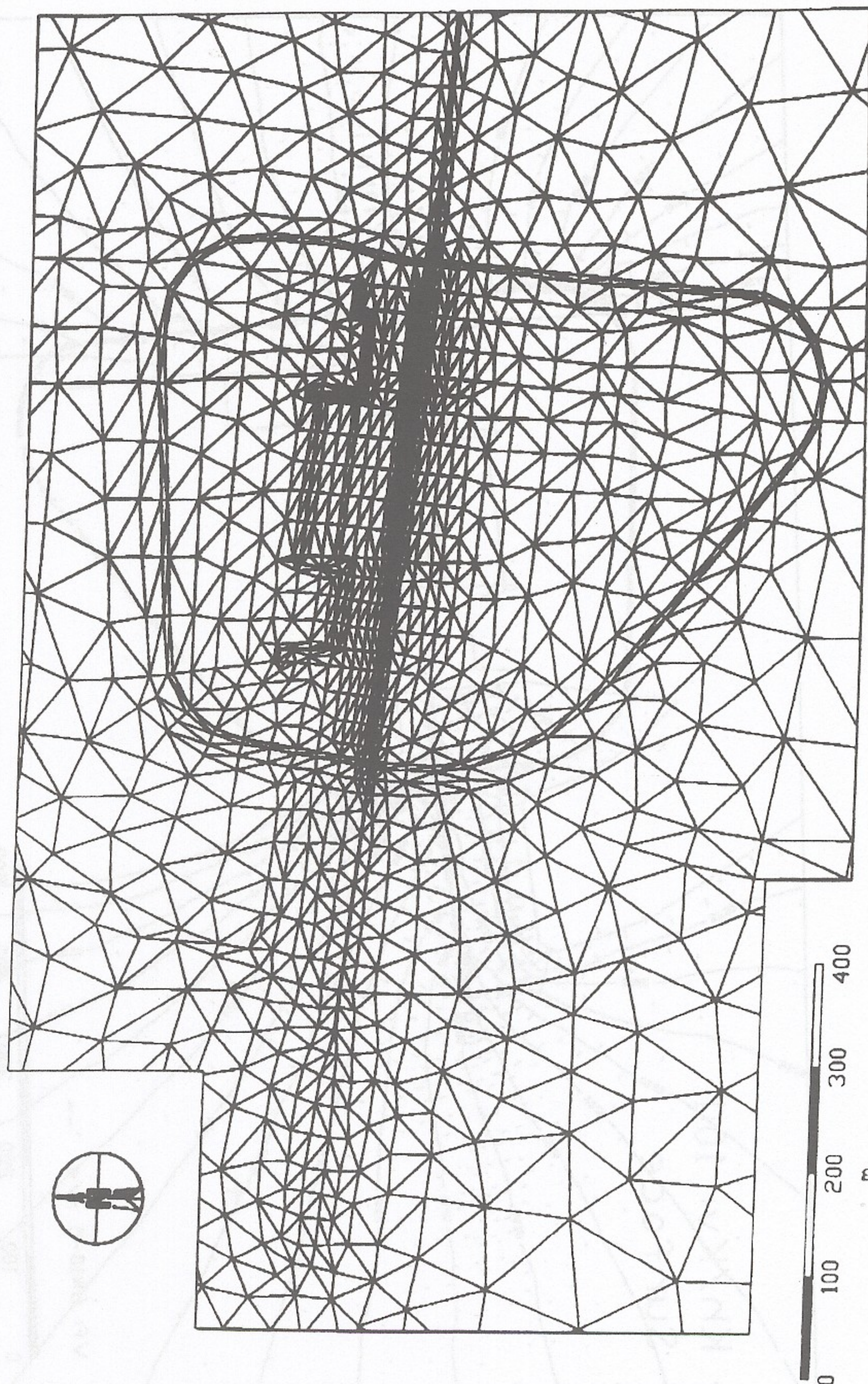
**A 7-2.3: Horizontal Discretization - Details**

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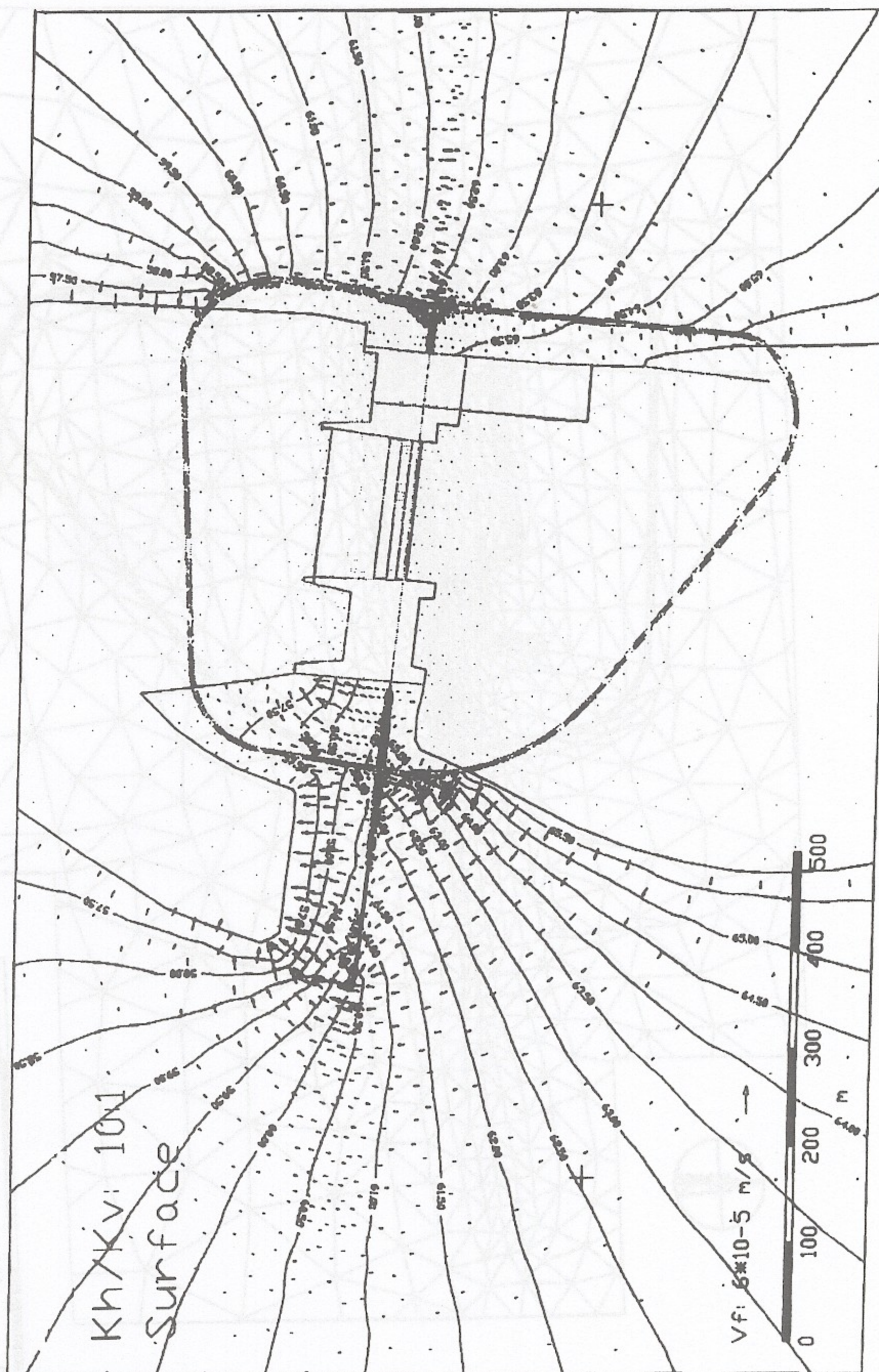
A 7-2.4: Calculated Water Table, $k_h / k_v = 10$

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**A 7-2.5: Seepage Underneath Diversion Canal Closure Embankment**

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A 7-2.6: Seepage near Left Powerhouse Corner

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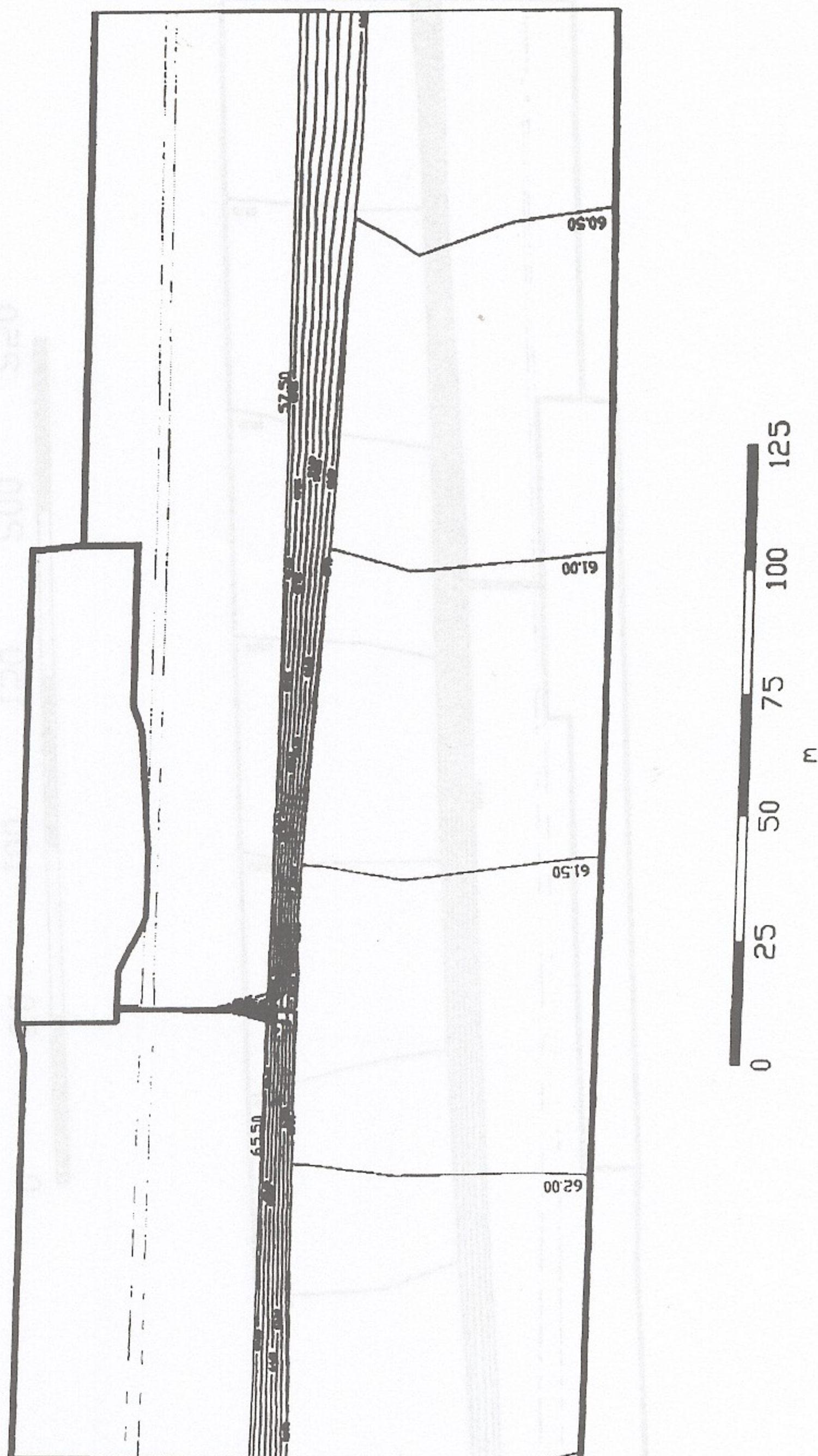
**A 7-2.7: Sluiceway Cut-off to Clay Layer**

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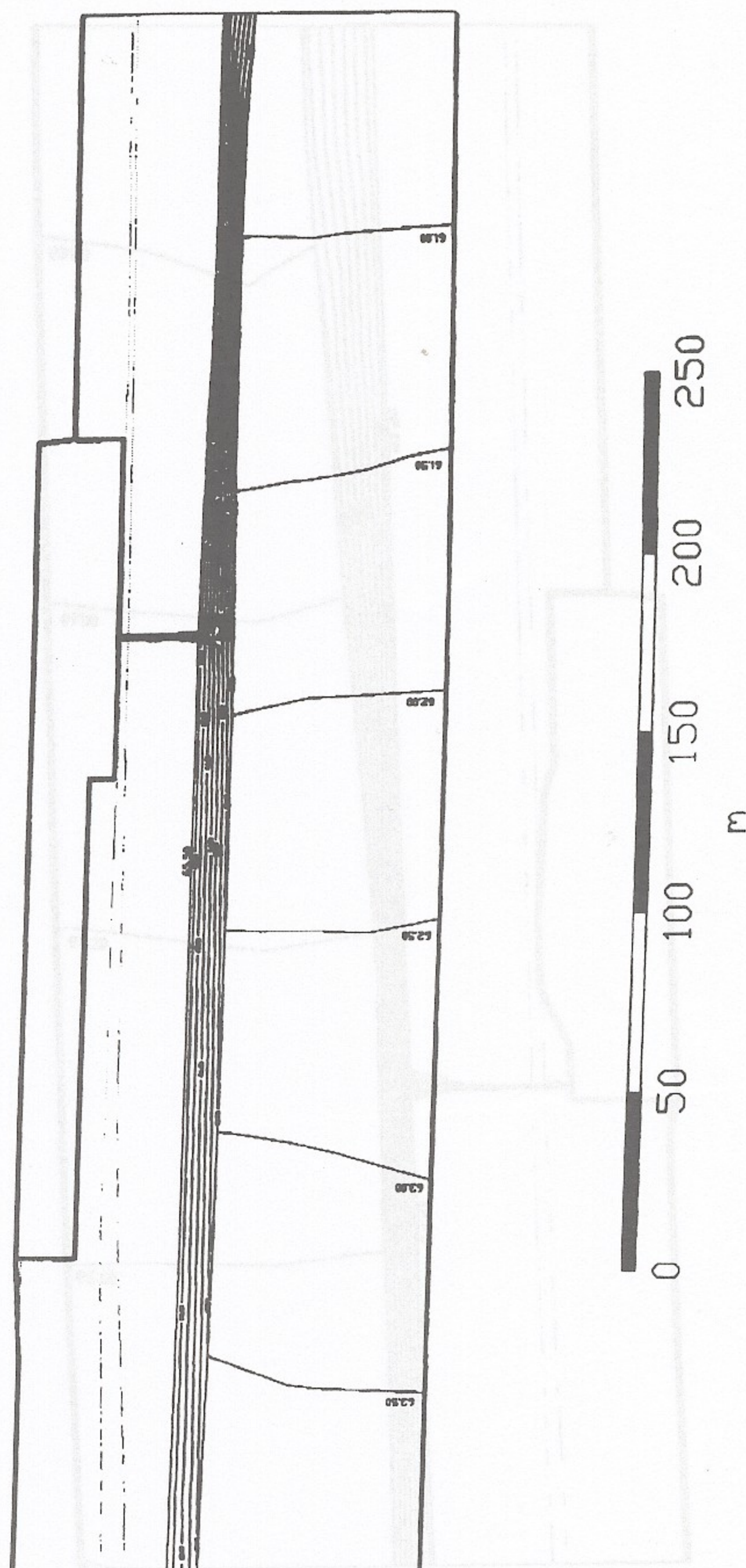
**A 7-2.8: Navigation Lock Cut-off to Clay Layer**

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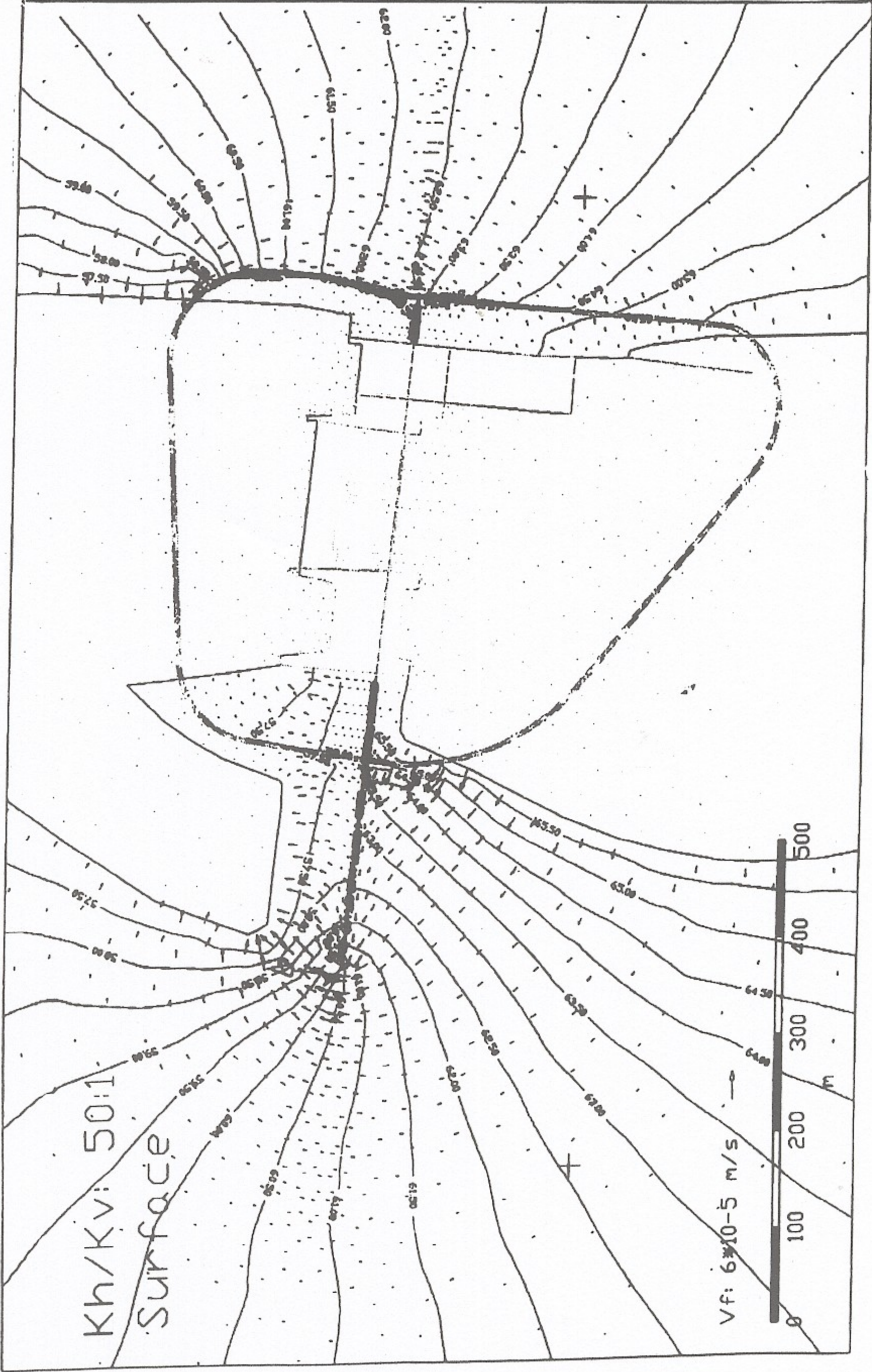
**A 7-2.9: Calculated Water Table, $k_h / k_v = 50$**

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# Appendix 7.3

# Construction Pit - Safety against Uplift

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## A 7-3  CONSTRUCTION PIT - SAFETY AGAINST UPLIFT

### A 7.3-1  GENERAL

For the tentative design of the New Naga Hammadi Barrage construction pit, calculations of the safety against uplift were carried out. The construction pit has a length of up 580 m and a width of some 530 m. The structural components of the barrage will be founded on different levels as given in **Table A 7.3-1** and as shown on **Album Drawing No. 18**.

### Table A 7.3-1:  Foundation Levels of Concrete Structures

| Structure | Excavation Level m asl |
|---|---|
| Powerhouse Abutment Pier | 38.00 to 42.40 |
| Powerhouse | 38.00 to 42.40 |
| Intermediate Pier:<br>- general<br>- oil separator/drainage sump | 38.00 to 42.40<br>35.90 |
| Sluiceway | 41.90 to 47.50 |
| Sluiceway Abutment Pier | 43.50 |
| Navigation Locks | 43.50 to 50.50 |

The bottom of the excavated and dewatered construction pit must be safe against uplift under the maximum drawdown of groundwater in the pit prior to the placement of concrete. The maximum drawdown will be required for the foundation of the powerhouse at 38.00 m asl, with a lower most water level of about 37.00 m asl. A further limited excavation down to 35.90 m asl for the drainage sump an area of some 20 m · 20 m will require temporary drawdown of the water level to 35.40 m asl as long as the foundation has been concreted.

Decisive for the safety against uplift is the weight of the sand and clay layer above the base of the clay layer, at which it is assumed that the hydrostatic water pressure acts uplifting. The lower the base of the clay layer, the higher is the weight and hence the factor of safety.

### A 7.3-2  PARAMETERS

#### A 7.3-2.1  Uplift

The hydrostatic water pressure below the clay layer is determined through the water level of the river Nile at the construction pit, except there would remain an artesian pressure which could result from the headpond of the old Barrage some 3 km upstream. The river water levels during the year are as given in **Table A 7.3-2**. The water for the diversion design flood of 2,900 m³/s passing the diversion canal is 62.48 m asl.

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Artesian water pressure induced from the old Barrage headpond through gravel layers of high permeability could affect the piezometric head below the clay layer. With the observations made at some of the drillings between the location of the New and the Old Barrage, it has been assumed that the uplift is governed by the river water level and by a remaining piezometric head of about 45% of the level differential between headpond and river at the construction site. For the design flood level of 62.48 m asl and a presently limited headpond level of less than 65.10 m asl, this results in a maximum expectable head of $(65.50 - 62.48) \cdot 0.45 + 62.48 = 63.84$ m asl or approximately 64.00 m asl.

**Table A 7.3-2: Average River Water Levels D/S of the Old Naga Hammadi Barrage**

| Month | Average<br>m asl | Maximum 10-Day<br>m asl | Minimum 10-Day<br>m asl |
|---|---|---|---|
| January | 58.71 | 59.14 | 57.86 |
| February | 59.61 | 59.68 | 59.46 |
| March | 60.19 | 60.41 | 59.86 |
| April | 60.21 | 60.27 | 60.17 |
| May | 60.59 | 60.96 | 60.25 |
| June | 61.68 | 61.72 | 61.63 |
| July | 61.56 | 61.59 | 61.51 |
| August | 61.25 | 61.36 | 61.10 |
| September | 60.32 | 60.75 | 59.78 |
| October | 59.58 | 59.66 | 59.53 |
| November | 59.71 | 59.99 | 59.35 |
| December | 58.81 | 58.85 | 58.75 |

**A 7.3-2.2   Geotechnical Properties**

Geotechnical properties were assumed based on the results of the geotechnical investigations (see **Volume 4** of this Tender Design Report) in accordance with EAB, §10.5 (EAB62), i.e. reduced parameters according to DIN 1055, Part 2, Tables 1 and 2. The properties are summarised in **Table A 7.3-3**.

**Table A 7.3-3:   Geotechnical Properties**

| Material | Weight acc. to DIN 1055 kN/m³ | Reduced Weight for Uplift Calculation kN/m³ |
|---|---|---|
| Sand (soil group SE), dense<br>- bulk<br>- uplift | <br>19<br>11 | <br>17<br>10 |
| Silt & clay (soil group TA), very stiff | 10 | 9 |

## A 7.3-2.3  In-situ Conditions

From the relevant borelogs after drillings around the deepest portion of the construction pit (see **Figure A 7-3.1**), the levels of the silt & clay layer are given in **Table A 7.3-4**. For the calculations of the safety factor, the generalising but unfavourable assumption was made that the bottom of the clay layer would be at 5.0 m asl and the top of the clay layer at 14.0 m asl. In addition the factor of safety was calculated for each relevant borehole with the actual vertical extensions of the silt & clay layer found.

### Table A 7.3-4: Vertical Extension of Silt & Clay Layer

| Area for Structure | Borehole | Silt & Clay Layer | |
| | | Bottom m asl | Top m asl |
|---|---|---|---|
| **Powerhouse** | | | |
| | 26 | 2.00 | 14.05 |
| | R9 | < 4.55 | 15.00 |
| | 2 | < 2.30 | 11.65 |
| | R12 | < 3.93 | 13.38 |
| | R11 | < 4.60 | 15.30 |
| | R8 | < 5.10 | 17.30 |
| | 9 | < 10.60 | 16.00 |
| **Sluiceway** | | | |
| | 9 | < 10.60 | 16.00 |
| | R8 | < 5.10 | 17.30 |
| | R11 | < 4.60 | 15.30 |
| | R10 | < 9.60 | 16.20 |
| | 5 | < 10.80 | 20.50 |
| | R7 | 8.80 | 17.50 |
| **Navigation Locks** | | | |
| | 5 | < 10.80 | 20.50 |
| | B10 | 12.10 | 21.50 |
| | 31 | 12.20 | 18.00 |

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## A 7.3-3 CALCULATION OF UPLIFT

The required factor of safety against uplift for the construction pit according to EAB is 1.10. For the calculations, it was assumed that the water level in the construction pit will be held at 1.0 m below the respective excavation level.

The results of the calculations are given in **Figures A 7.3-2** to **7.3-14**. The calculations with the simplified unfavourable assumptions are summarised in **Figure A 7.3-2**. In general it can be stated, that in the powerhouse foundation area excavations down to elevation 38.0 m asl are always giving a factor of safety above 1.10 (**Figures A 7.3-3** to **7.3-9**). Critical remains the small area of excavation below 38.0 m asl for the drainage sump under the intermediate pier at the powerhouse. Applying the geotechnical results of borehole BH26, which is the borehole closest to the investigated area, the required factor of safety 1.10 is achieved down to elevation 36.70 m asl (**Figures A 7.3-3**).

Excavations for the sluiceway and its piers (**Figures A 7.3-7** to **7.12**)and for the navigation locks (**Figures A 7.3-10** to **7.14**) always give factors of safety higher than 1.10

To achieve the required factor of safety for an excavation level of 35.90 m asl, the piezometric head shall be at maximum 62.25 m asl (**Figure A 7.3-15**). Therefore, excavation and concreting works for this lowest part of the powerhouse has to be executed during the low flow season. The works will require about 2 months and could be accommodated in the period from October to Februray inclusive, when the river water level at the construction site will remain below 59.9 m asl.

It is further required that during excavation the Contractor performs drillings in order to measure the piezometric head prevailing below the clay layer. This will allow control of the safety against uplift when approaching the deepest part of the construction pit by excavation.
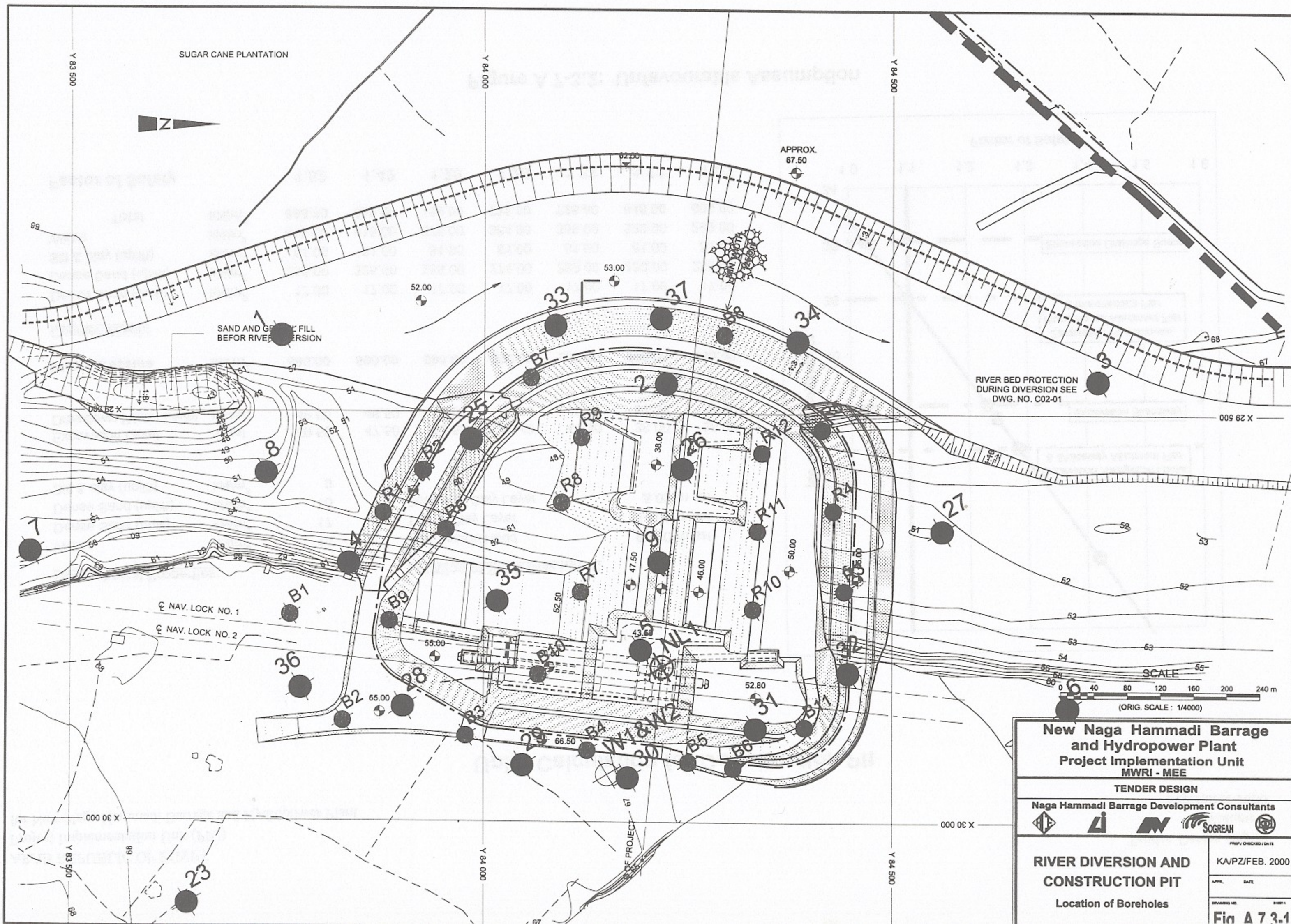
SUGAR CANE PLANTATION

N

APPROX.
67.50

SAND AND GRAVEL FILL
BEFOR RIVER DIVERSION

RIVER BED PROTECTION
DURING DIVERSION SEE
DWG. NO. C02-01

NAV. LOCK NO. 1

NAV. LOCK NO. 2

SCALE

0   40   80   120   160   200   240 m

(ORIG. SCALE : 1/4000)

**New Naga Hammadi Barrage
and Hydropower Plant**
**Project Implementation Unit**
MWRI - MEE

TENDER DESIGN

Naga Hammadi Barrage Development Consultants

SOGREAH

**RIVER DIVERSION AND
CONSTRUCTION PIT**

Location of Boreholes

KA/PZ/FEB. 2000

Fig. A 7.3-1

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# Uplift Calculation for Construction Pit

**Geotechnical Properties:**

| Property | Unit | Value |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| | | |
|---|---|---|
| Piezometric Head | 64.00 | m asl |
| Top of Clay Layer | 14.00 | m asl |
| Bottom of Clay Layer | 5.00 | m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| **Water pressure** | kN/m$^2$ | 590.00 | 590.00 | 590.00 | 590.00 | 590.00 | 590.00 | 590.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 355.00 | 325.00 | 285.00 | 274.00 | 269.00 | 230.00 | 209.00 |
| Silt & clay (uplift) | kN/m$^2$ | 81.00 | 81.00 | 81.00 | 81.00 | 81.00 | 81.00 | 81.00 |
| Water | kN/m$^2$ | 445.00 | 415.00 | 375.00 | 364.00 | 359.00 | 320.00 | 299.00 |
| **Total** | kN/m$^2$ | 898.00 | 838.00 | 758.00 | 736.00 | 726.00 | 648.00 | 606.00 |
| | | | | | | | | |
| **Factor of Safety** | | 1.52 | 1.42 | 1.28 | 1.25 | 1.23 | 1.10 | 1.03 |
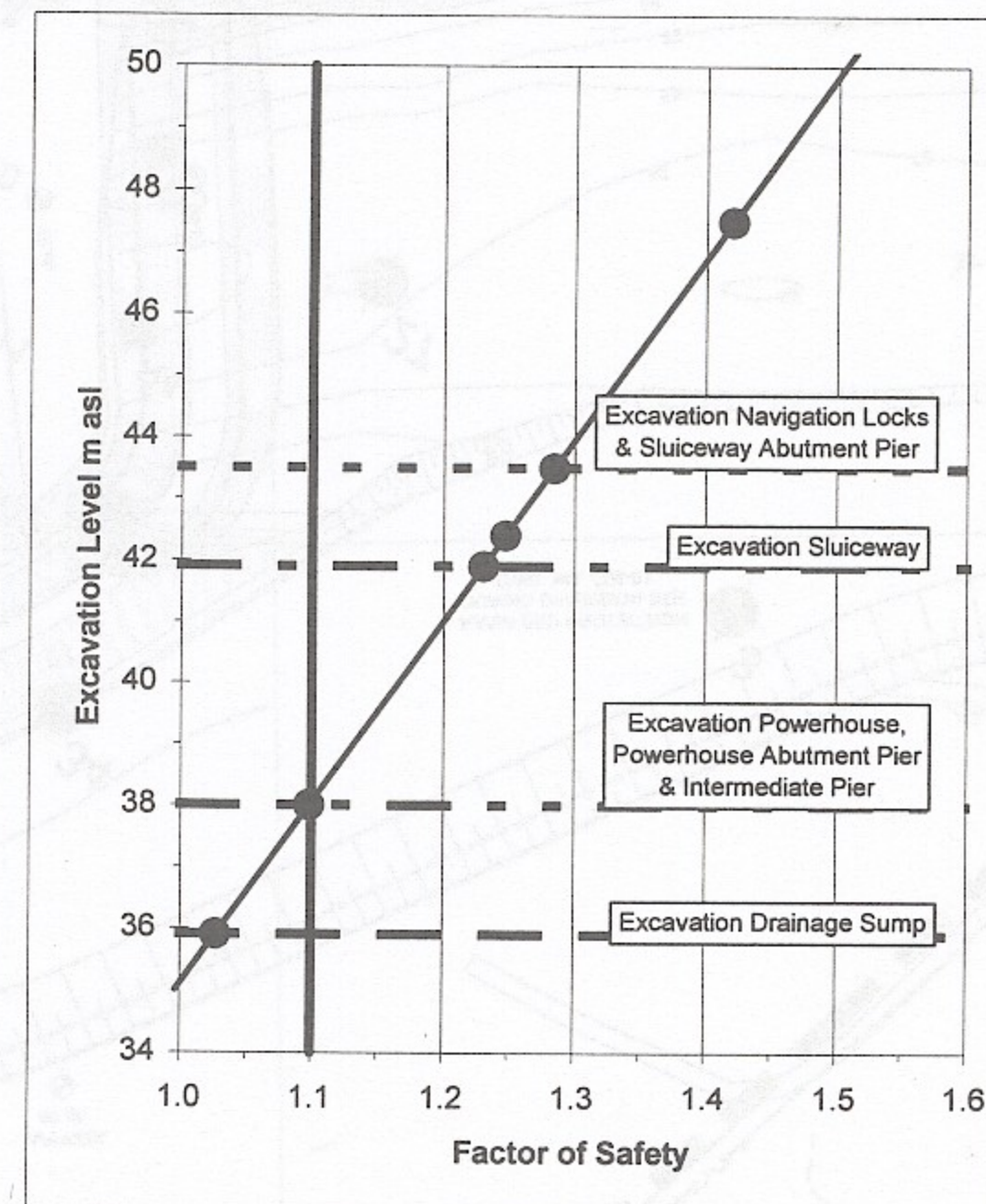


Figure A 7-3.2:  Unfavourable Assumption

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# Uplift Calculation for Construction Pit at Powerhouse Foundation

### Geotechnical Properties:

| Water | kN/m$^3$ | 10 |
|---|---|---|
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

### Location of Silt & Clay Layer:

| Piezometric Head | 64.00 m asl |
|---|---|
| Top of Clay Layer | 14.05 m asl |
| Bottom of Clay Layer | 2.00 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| Water pressure | kN/m$^2$ | 620.00 | 620.00 | 620.00 | 620.00 | 620.00 | 620.00 | 620.00 |

### Counter Weight

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 354.50 | 324.50 | 284.50 | 273.50 | 268.50 | 229.50 | 208.50 |
| Silt & clay (uplift) | kN/m$^2$ | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 |
| Water | kN/m$^2$ | 475.00 | 445.00 | 405.00 | 394.00 | 389.00 | 350.00 | 329.00 |
| Total | kN/m$^2$ | 954.95 | 894.95 | 814.95 | 792.95 | 782.95 | 704.95 | 662.95 |

| **Factor of Safety** | | **1.54** | **1.44** | **1.31** | **1.28** | **1.26** | **1.14** | **1.07** |
|---|---|---|---|---|---|---|---|---|



Figure A 7-3.3:  Borehole 26

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# Uplift Calculation for Construction Pit at Powerhouse & Abutment Pier Foundation

**Geotechnical Properties:**

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 11.65 m asl |
| Bottom of Clay Layer | 2.30 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| **Water pressure** | kN/m$^2$ | 617.00 | 617.00 | 617.00 | 617.00 | 617.00 | 617.00 | 617.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 378.50 | 348.50 | 308.50 | 297.50 | 292.50 | 253.50 | 232.50 |
| Silt & clay (uplift) | kN/m$^2$ | 84.15 | 84.15 | 84.15 | 84.15 | 84.15 | 84.15 | 84.15 |
| Water | kN/m$^2$ | 472.00 | 442.00 | 402.00 | 391.00 | 386.00 | 347.00 | 326.00 |
| Total | kN/m$^2$ | 951.65 | 891.65 | 811.65 | 789.65 | 779.65 | 701.65 | 659.65 |
| **Factor of Safety** | | **1.54** | **1.45** | **1.32** | **1.28** | **1.26** | **1.14** | **1.07** |



Figure A 7-3.5: Borehole 2

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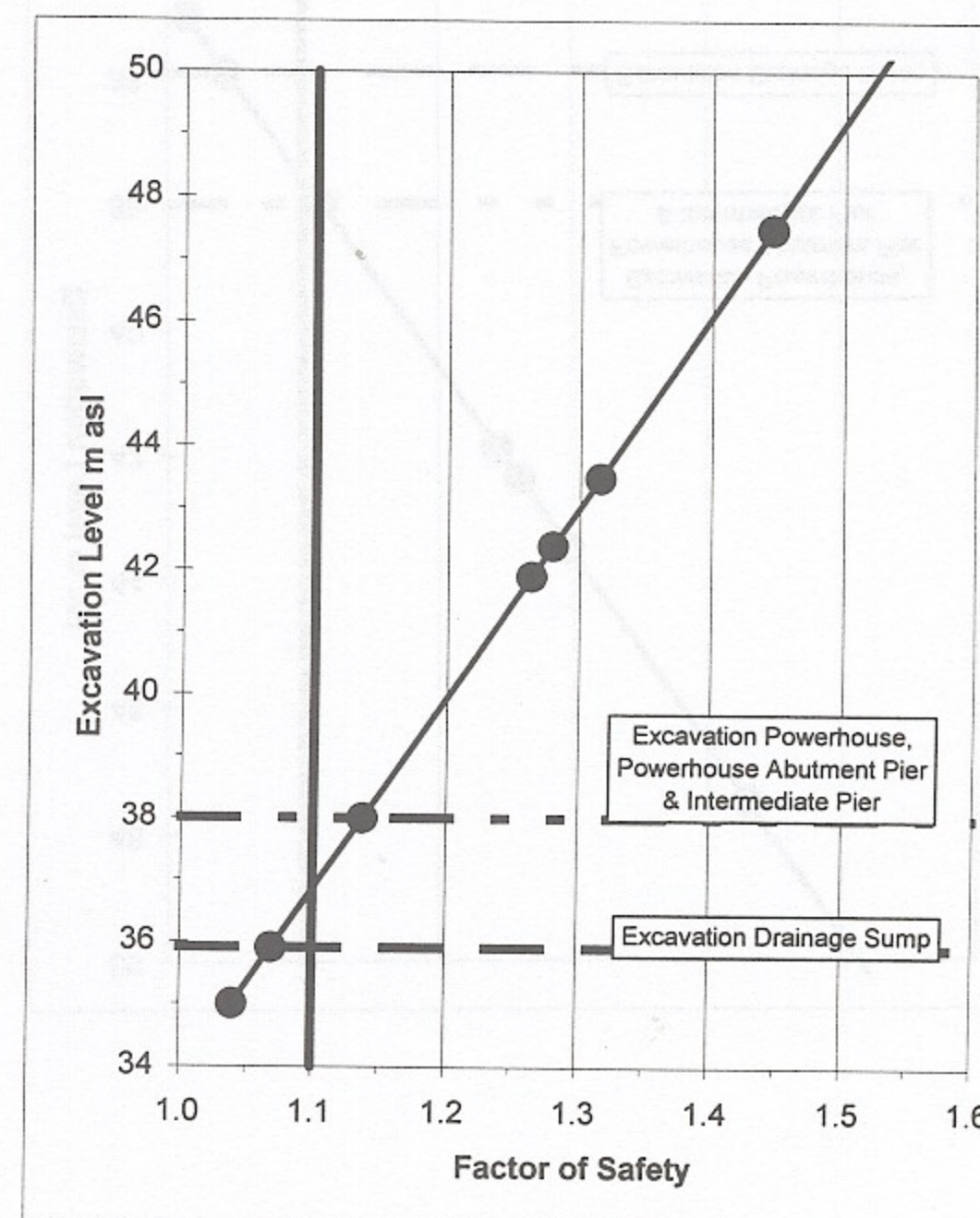
# Uplift Calculation for Construction Pit at D/S Powerhouse & Abutment Pier Foundation

**Geotechnical Properties:**

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 13.38 m asl |
| Bottom of Clay Layer | 3.93 m asl |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| Water pressure | kN/m$^2$ | 600.70 | 600.70 | 600.70 | 600.70 | 600.70 | 600.70 | 600.70 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 361.20 | 331.20 | 291.20 | 280.20 | 275.20 | 236.20 | 215.20 |
| Silt & clay (uplift) | kN/m$^2$ | 85.05 | 85.05 | 85.05 | 85.05 | 85.05 | 85.05 | 85.05 |
| Water | kN/m$^2$ | 455.70 | 425.70 | 385.70 | 374.70 | 369.70 | 330.70 | 309.70 |
| Total | kN/m$^2$ | 918.95 | 858.95 | 778.95 | 756.95 | 746.95 | 668.95 | 626.95 |
| **Factor of Safety** | | **1.53** | **1.43** | **1.30** | **1.26** | **1.24** | **1.11** | **1.04** |
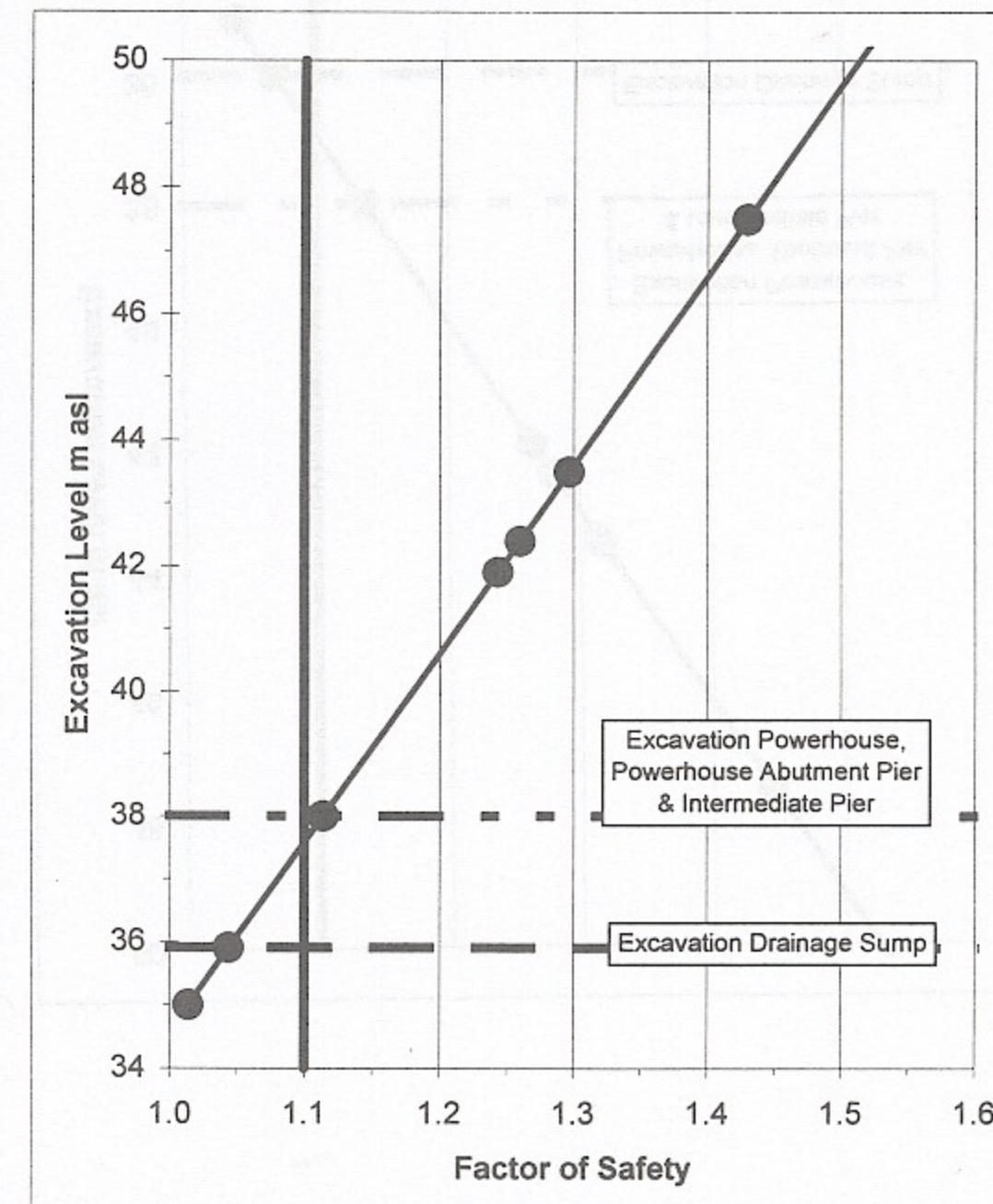


Figure A 7-3.6: Borehole R12

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# Uplift Calculation for Construction Pit at Intermediate Pier Foundation & D/S Sluiceway

<u>Geotechnical Properties:</u>

| Water | kN/m³ | 10 |
| Dense Sand (bulk) | kN/m³ | 17 |
| Dense Sand (uplift) | kN/m³ | 10 |
| Silt & clay (uplift) | kN/m³ | 9 |

<u>Location of Silt & Clay Layer:</u>

| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 15.30 m asl |
| Bottom of Clay Layer | 4.60 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| **Water pressure** | kN/m² | 594.00 | 594.00 | 594.00 | 594.00 | 594.00 | 594.00 | 594.00 |

<u>Counter Weight</u>

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m² | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m² | 342.00 | 312.00 | 272.00 | 261.00 | 256.00 | 217.00 | 196.00 |
| Silt & clay (uplift) | kN/m² | 96.30 | 96.30 | 96.30 | 96.30 | 96.30 | 96.30 | 96.30 |
| Water | kN/m² | 449.00 | 419.00 | 379.00 | 368.00 | 363.00 | 324.00 | 303.00 |
| Total | kN/m² | 904.30 | 844.30 | 764.30 | 742.30 | 732.30 | 654.30 | 612.30 |
| **Factor of Safety** | | 1.52 | 1.42 | 1.29 | 1.25 | 1.23 | 1.10 | 1.03 |
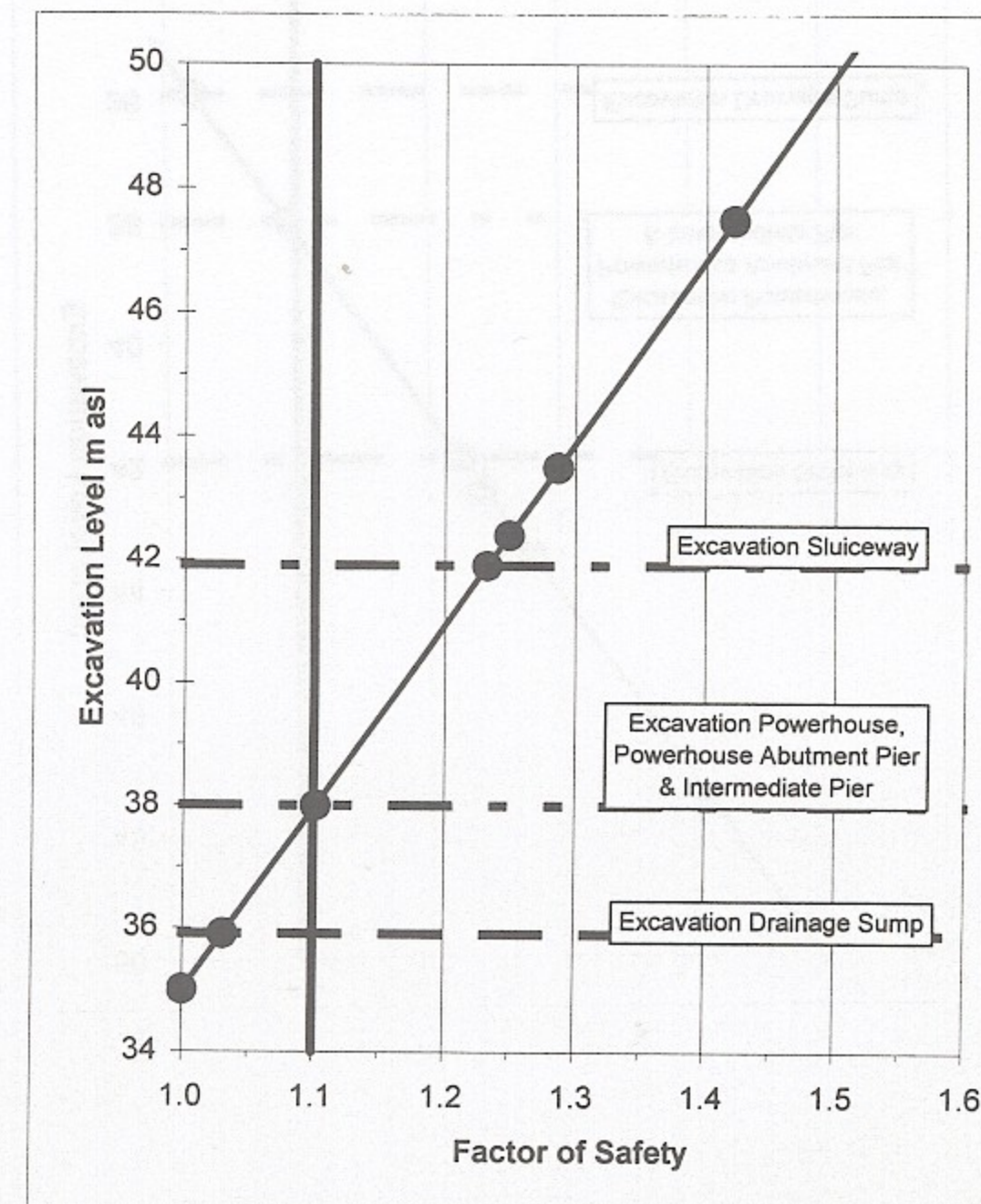


Figure A 7-3.7:  Borehole R11

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# Uplift Calculation for Construction Pit at Intermediate Pier Foundation & U/S Sluiceway

**Geotechnical Properties:**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Water | kN/m³ | 10 | | | | | | |
| Dense Sand (bulk) | kN/m³ | 17 | | | | | | |
| Dense Sand (uplift) | kN/m³ | 10 | | | | | | |
| Silt & clay (uplift) | kN/m³ | 9 | | | | | | |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 17.30 m asl |
| Bottom of Clay Layer | 5.10 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| **Water pressure** | kN/m² | 589.00 | 589.00 | 589.00 | 589.00 | 589.00 | 589.00 | 589.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m² | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m² | 322.00 | 292.00 | 252.00 | 241.00 | 236.00 | 197.00 | 176.00 |
| Silt & clay (uplift) | kN/m² | 109.80 | 109.80 | 109.80 | 109.80 | 109.80 | 109.80 | 109.80 |
| Water | kN/m² | 444.00 | 414.00 | 374.00 | 363.00 | 358.00 | 319.00 | 298.00 |
| Total | kN/m² | 892.80 | 832.80 | 752.80 | 730.80 | 720.80 | 642.80 | 600.80 |
| **Factor of Safety** | | **1.52** | **1.41** | **1.28** | **1.24** | **1.22** | **1.09** | **1.02** |



Figure A 7-3.8:  Borehole R8

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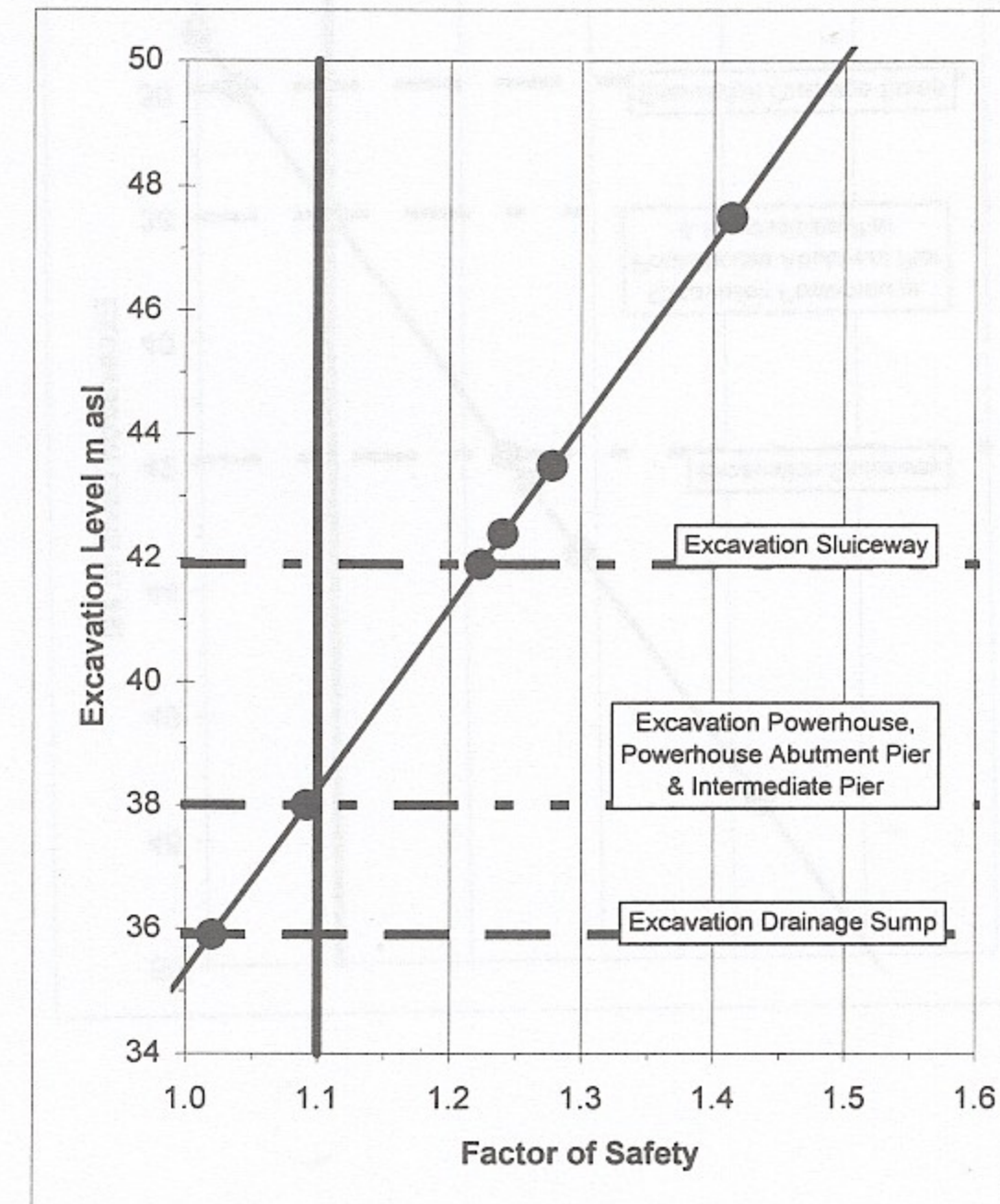
# Uplift Calculation for Construction Pit at Intermediate Pier & Sluiceway Foundation

**Geotechnical Properties:**

| Water | kN/m$^3$ | 10 |
|---|---|---|
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| Piezometric Head | 64.00 m asl |
|---|---|
| Top of Clay Layer | 16.00 m asl |
| Bottom of Clay Layer | 10.60 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| Water pressure | kN/m$^2$ | 534.00 | 534.00 | 534.00 | 534.00 | 534.00 | 534.00 | 534.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 335.00 | 305.00 | 265.00 | 254.00 | 249.00 | 210.00 | 189.00 |
| Silt & clay (uplift) | kN/m$^2$ | 48.60 | 48.60 | 48.60 | 48.60 | 48.60 | 48.60 | 48.60 |
| Water | kN/m$^2$ | 389.00 | 359.00 | 319.00 | 308.00 | 303.00 | 264.00 | 243.00 |
| Total | kN/m$^2$ | 789.60 | 729.60 | 649.60 | 627.60 | 617.60 | 539.60 | 497.60 |
| **Factor of Safety** | | **1.48** | **1.37** | **1.22** | **1.18** | **1.16** | **1.01** | **0.93** |



Figure A 7-3.9:  Borehole 9

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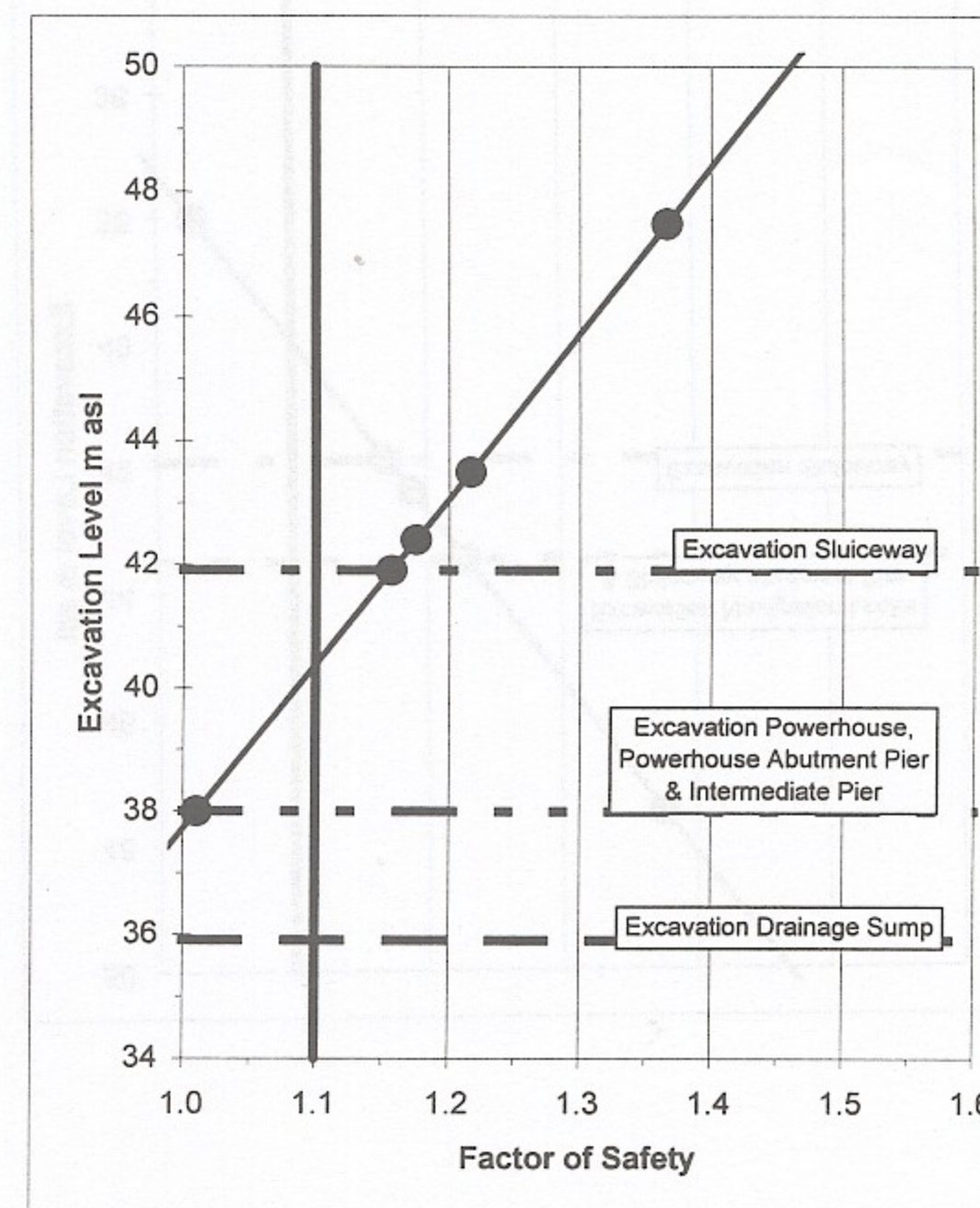
# Uplift Calculation for Construction Pit at D/S Sluiceway & Sluiceway Abutment Pier Foundation

**Geotechnical Properties:**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Water | kN/m³ | 10 | | | | | | |
| Dense Sand (bulk) | kN/m³ | 17 | | | | | | |
| Dense Sand (uplift) | kN/m³ | 10 | | | | | | |
| Silt & clay (uplift) | kN/m³ | 9 | | | | | | |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 16.20 m asl |
| Bottom of Clay Layer | 9.60 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| **Water pressure** | kN/m² | 544.00 | 544.00 | 544.00 | 544.00 | 544.00 | 544.00 | 544.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m² | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m² | 333.00 | 303.00 | 263.00 | 252.00 | 247.00 | 208.00 | 187.00 |
| Silt & clay (uplift) | kN/m² | 59.40 | 59.40 | 59.40 | 59.40 | 59.40 | 59.40 | 59.40 |
| Water | kN/m² | 399.00 | 369.00 | 329.00 | 318.00 | 313.00 | 274.00 | 253.00 |
| Total | kN/m² | 808.40 | 748.40 | 668.40 | 646.40 | 636.40 | 558.40 | 516.40 |
| | | | | | | | | |
| **Factor of Safety** | | 1.49 | 1.38 | 1.23 | 1.19 | 1.17 | 1.03 | 0.95 |
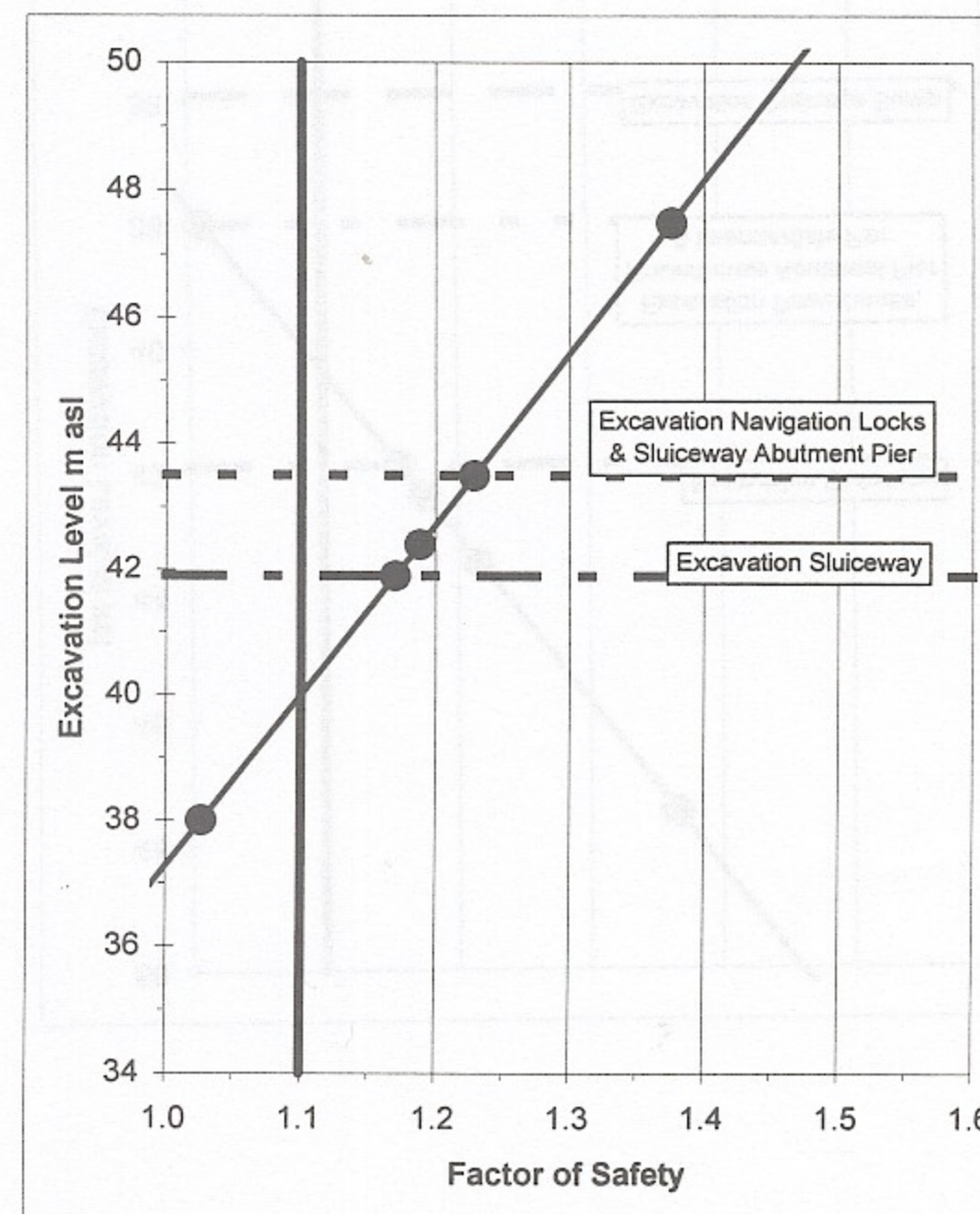


Figure A 7-3.10: Borehole R10

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# Uplift Calculation for Construction Pit at U/S Sluiceway & Sluiceway Abutment Pier Foundation

**Geotechnical Properties:**

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 17.50 m asl |
| Bottom of Clay Layer | 8.80 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| Water pressure | kN/m$^2$ | 552.00 | 552.00 | 552.00 | 552.00 | 552.00 | 552.00 | 552.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 320.00 | 290.00 | 250.00 | 239.00 | 234.00 | 195.00 | 174.00 |
| Silt & clay (uplift) | kN/m$^2$ | 78.30 | 78.30 | 78.30 | 78.30 | 78.30 | 78.30 | 78.30 |
| Water | kN/m$^2$ | 407.00 | 377.00 | 337.00 | 326.00 | 321.00 | 282.00 | 261.00 |
| Total | kN/m$^2$ | 822.30 | 762.30 | 682.30 | 660.30 | 650.30 | 572.30 | 530.30 |
| **Factor of Safety** | | 1.49 | 1.38 | 1.24 | 1.20 | 1.18 | 1.04 | 0.96 |
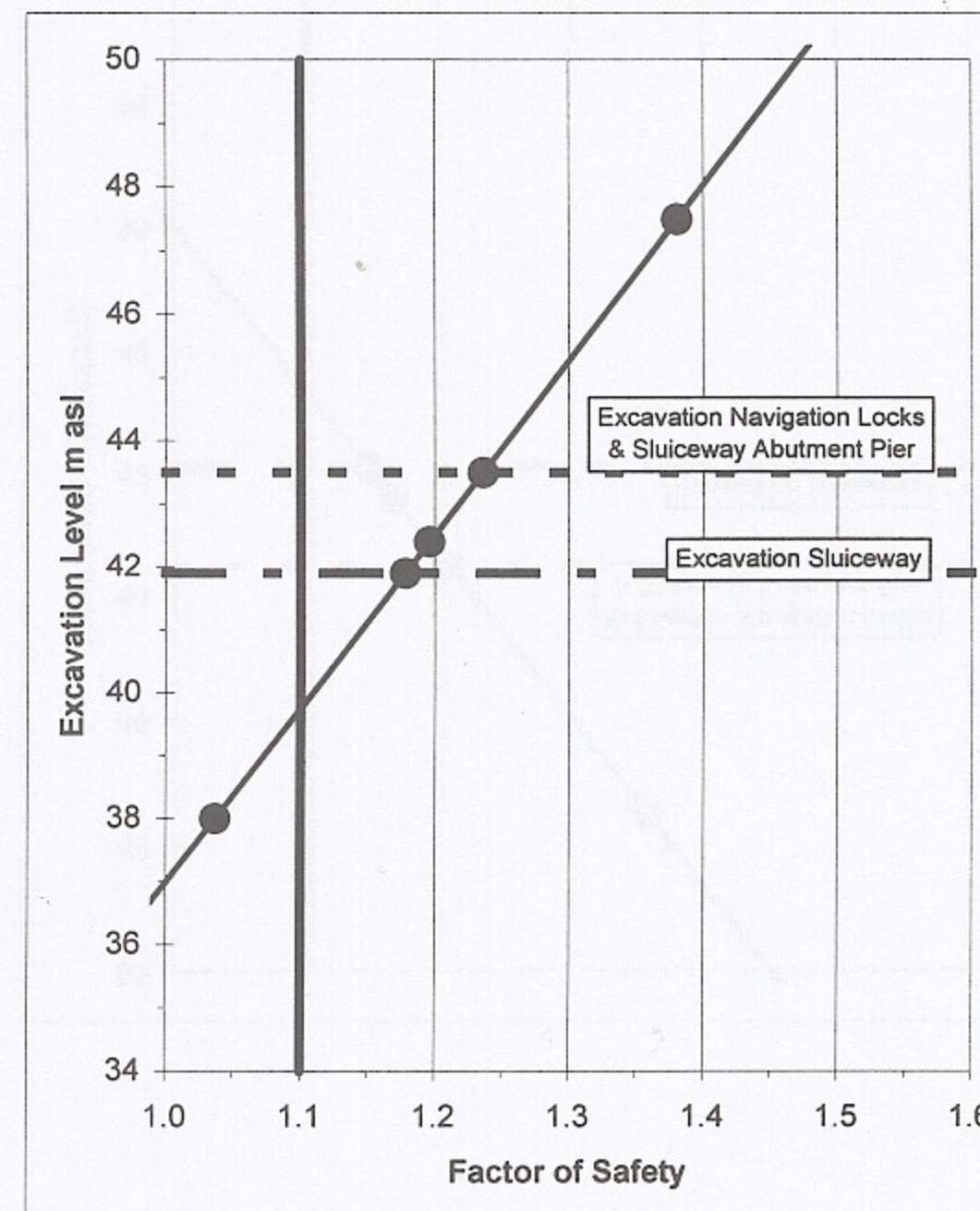


Figure A 7-3.11: Borehole R7

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# Uplift Calculation for Construction Pit at Sluiceway Abutment Pier & Navigation Locks Foundations

## Geotechnical Properties:

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

## Location of Silt & Clay Layer:

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 20.50 m asl |
| Bottom of Clay Layer | 10.80 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| **Water pressure** | kN/m$^2$ | 532.00 | 532.00 | 532.00 | 532.00 | 532.00 | 532.00 | 532.00 |

## Counter Weight

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 290.00 | 260.00 | 220.00 | 209.00 | 204.00 | 165.00 | 144.00 |
| Silt & clay (uplift) | kN/m$^2$ | 87.30 | 87.30 | 87.30 | 87.30 | 87.30 | 87.30 | 87.30 |
| Water | kN/m$^2$ | 387.00 | 357.00 | 317.00 | 306.00 | 301.00 | 262.00 | 241.00 |
| Total | kN/m$^2$ | 781.30 | 721.30 | 641.30 | 619.30 | 609.30 | 531.30 | 489.30 |
| **Factor of Safety** | | **1.47** | **1.36** | **1.21** | **1.16** | **1.15** | **1.00** | **0.92** |



Figure A 7-3.12: Borehole 5

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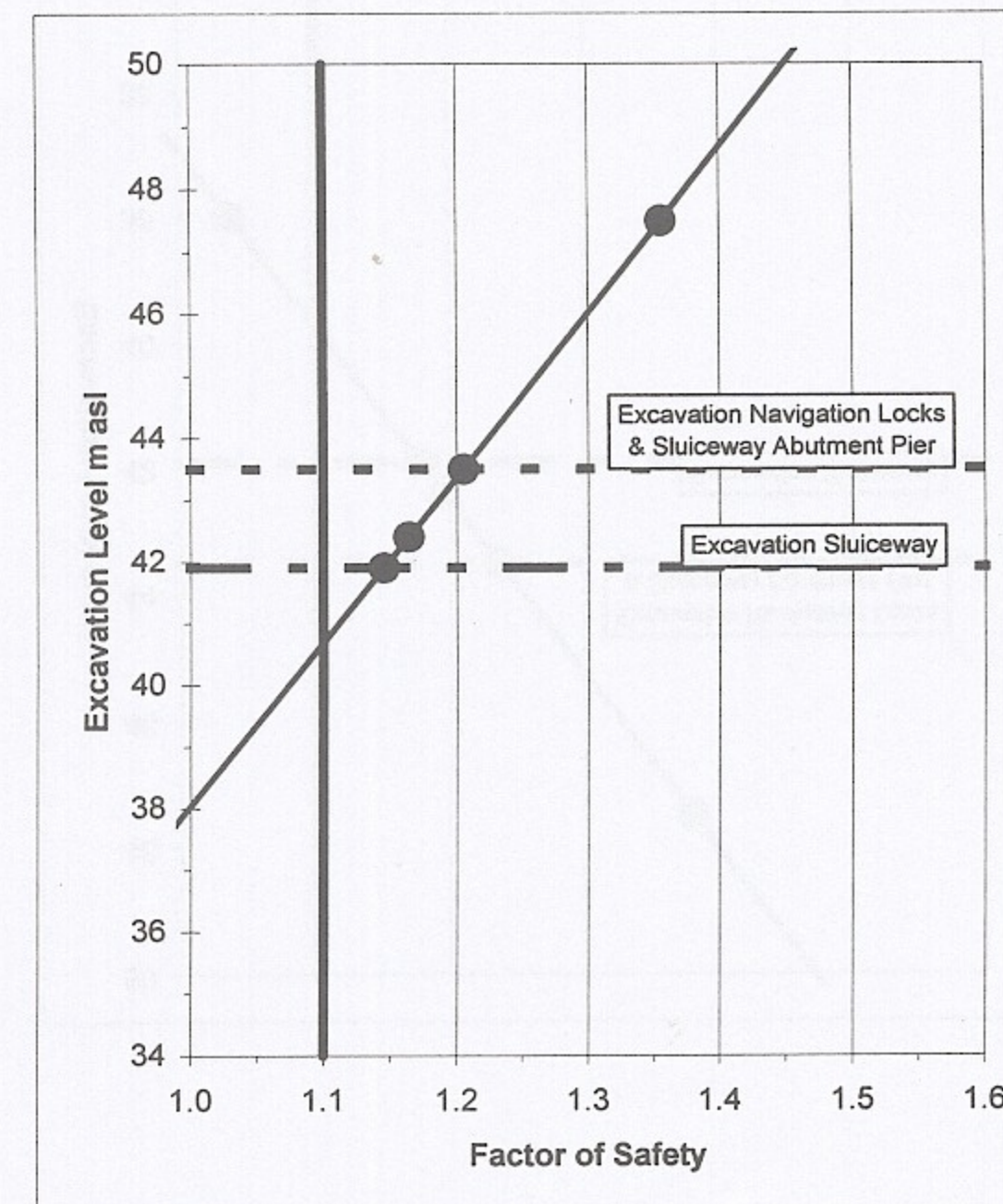
# Uplift Calculation for Construction Pit at US Navigation Locks Foundation

## Geotechnical Properties:

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

## Location of Silt & Clay Layer:

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 21.50 m asl |
| Bottom of Clay Layer | 12.10 m asl |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Water pressure** | kN/m$^2$ | 519.00 | 519.00 | 519.00 | 519.00 | 519.00 | 519.00 | 519.00 |

## Counter Weight

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 280.00 | 250.00 | 210.00 | 199.00 | 194.00 | 155.00 | 134.00 |
| Silt & clay (uplift) | kN/m$^2$ | 84.60 | 84.60 | 84.60 | 84.60 | 84.60 | 84.60 | 84.60 |
| Water | kN/m$^2$ | 374.00 | 344.00 | 304.00 | 293.00 | 288.00 | 249.00 | 228.00 |
| Total | kN/m$^2$ | 755.60 | 695.60 | 615.60 | 593.60 | 583.60 | 505.60 | 463.60 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Factor of Safety** | **1.46** | **1.34** | **1.19** | **1.14** | **1.12** | **0.97** | **0.89** |
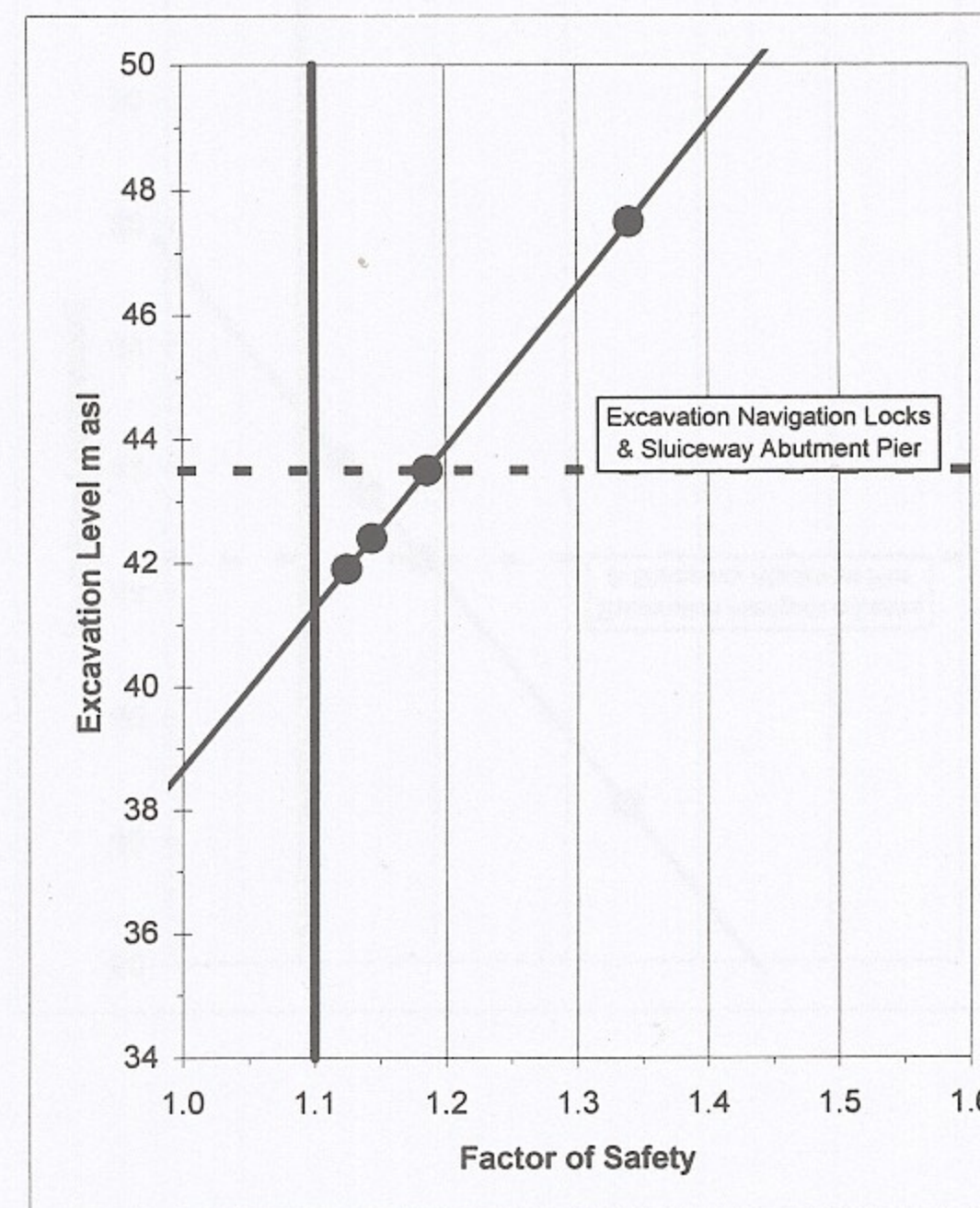


Figure A 7-3.13: Borehole B10

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# Uplift Calculation for Construction Pit at DS Navigation Locks Foundation

### Geotechnical Properties:

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

### Location of Silt & Clay Layer:

| | |
|---|---|
| Piezometric Head | 64.00 m asl |
| Top of Clay Layer | 18.00 m asl |
| Bottom of Clay Layer | 12.20 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Excavation Level | m asl | 50.50 | 47.50 | 43.50 | 42.40 | 41.90 | 38.00 | 35.90 |
| Drawdown Water Level | m asl | 49.50 | 46.50 | 42.50 | 41.40 | 40.90 | 37.00 | 34.90 |
| | | | | | | | | |
| **Water pressure** | kN/m$^2$ | 518.00 | 518.00 | 518.00 | 518.00 | 518.00 | 518.00 | 518.00 |

### Counter Weight

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 315.00 | 285.00 | 245.00 | 234.00 | 229.00 | 190.00 | 169.00 |
| Silt & clay (uplift) | kN/m$^2$ | 52.20 | 52.20 | 52.20 | 52.20 | 52.20 | 52.20 | 52.20 |
| Water | kN/m$^2$ | 373.00 | 343.00 | 303.00 | 292.00 | 287.00 | 248.00 | 227.00 |
| Total | kN/m$^2$ | 757.20 | 697.20 | 617.20 | 595.20 | 585.20 | 507.20 | 465.20 |
| **Factor of Safety** | | **1.46** | **1.35** | **1.19** | **1.15** | **1.13** | **0.98** | **0.90** |
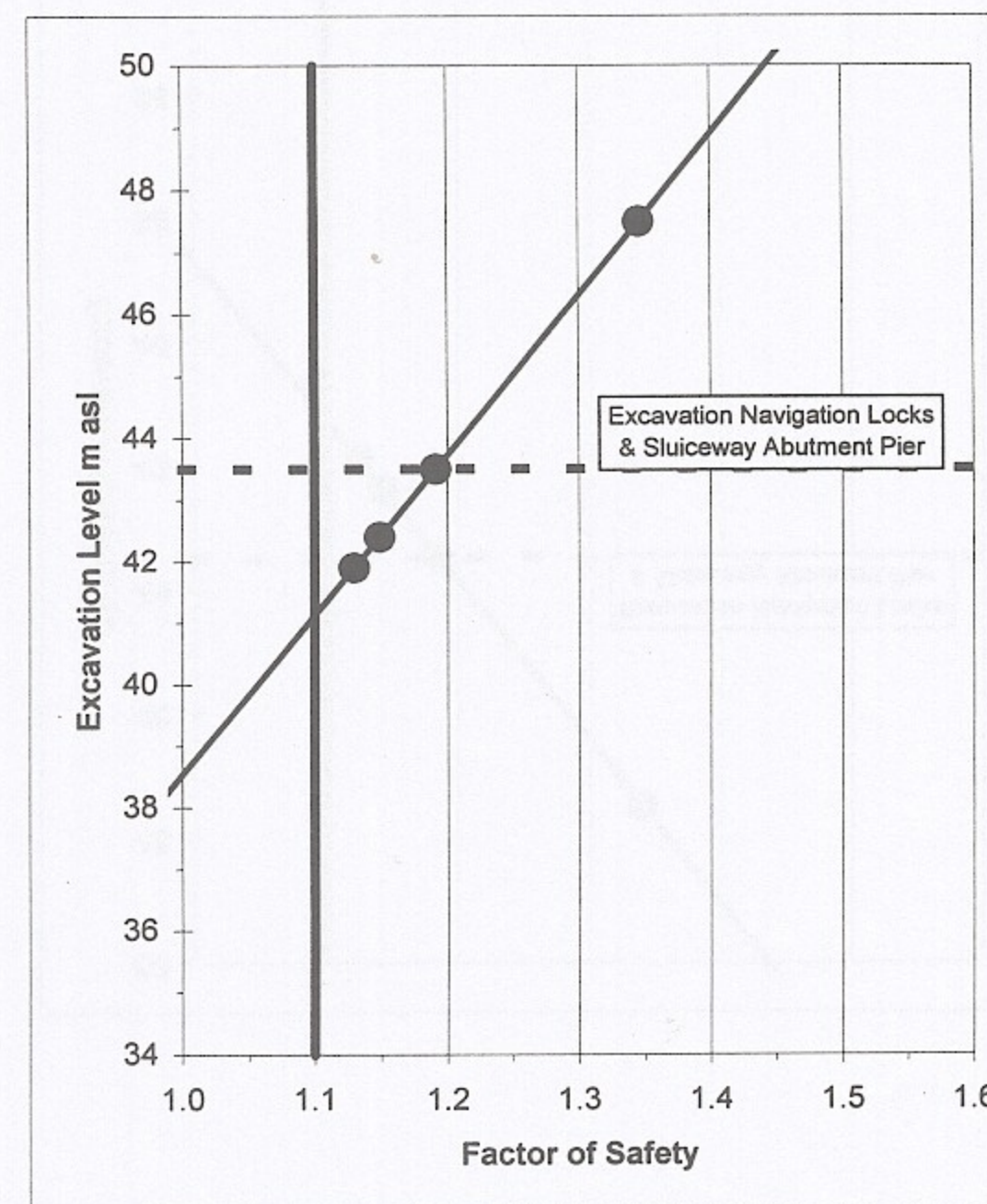


**Figure A 7-3.14: Borehole 31**

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## Uplift Calculation for Construction Pit at Powerhouse Foundation for Drainage Sump as f (piezometric head

**Geotechnical Properties:**

| | | |
|---|---|---|
| Water | kN/m$^3$ | 10 |
| Dense Sand (bulk) | kN/m$^3$ | 17 |
| Dense Sand (uplift) | kN/m$^3$ | 10 |
| Silt & clay (uplift) | kN/m$^3$ | 9 |

**Location of Silt & Clay Layer:**

| | |
|---|---|
| Excavation Level | 35.90 m asl |
| Top of Clay Layer | 14.05 m asl |
| Bottom of Clay Layer | 2.00 m asl |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Piezometric Head | m asl | 64.00 | 63.50 | 63.00 | 62.50 | 62.25 | 62.00 | 61.00 |
| Drawdown Water Level | m asl | 34.90 | 34.90 | 34.90 | 34.90 | 34.90 | 34.90 | 34.90 |
| Water pressure | kN/m$^2$ | 620.00 | 615.00 | 610.00 | 605.00 | 602.50 | 600.00 | 590.00 |

**Counter Weight**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Dense Sand (bulk) | kN/m$^2$ | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 | 17.00 |
| Dense Sand (uplift) | kN/m$^2$ | 208.50 | 208.50 | 208.50 | 208.50 | 208.50 | 208.50 | 208.50 |
| Silt & clay (uplift) | kN/m$^2$ | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 | 108.45 |
| Water | kN/m$^2$ | 329.00 | 329.00 | 329.00 | 329.00 | 329.00 | 329.00 | 329.00 |
| Total | kN/m$^2$ | 662.95 | 662.95 | 662.95 | 662.95 | 662.95 | 662.95 | 662.95 |
| **Factor of Safety** | | **1.069** | **1.078** | **1.087** | **1.096** | **1.100** | **1.105** | **1.124** |
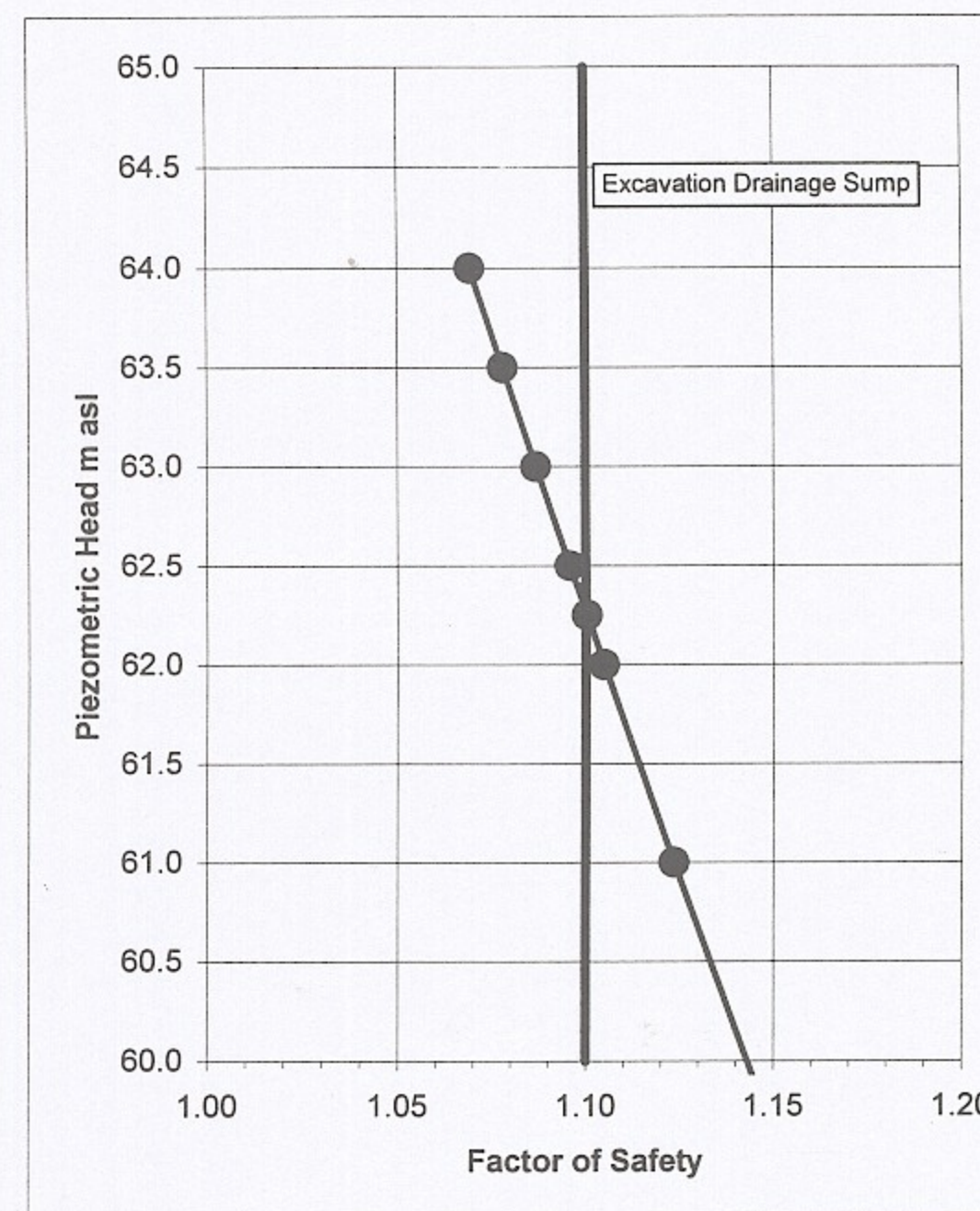


Figure A 7-3.15: Borehole 26

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# CHAPTER 8

# REHABILITATION AND UPGRADING OF HEAD REGULATORS

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# 8.    REHABILITATION AND UPGRADING OF HEAD REGULATORS

## 8.1    Structures, Maintenance and Operation

There are two major irrigation canals, one on each side of the River Nile branching off upstream of the Naga Hammadi Barrage:

- the Fouadia Canal on the west;

- the Faroukia Canal on the east.

They were built at the same time as the Naga Hammadi Barrage, in the years 1927 to 1930. At the head of each canal a weir, called "Head Regulator", is arranged to feed and to control the discharge into the canals. In addition both head regulators serve as bridge over the respective canal.

The Fouadia or Western head regulator is situated on the left side of the Nile and has 6 gated openings, while the Faroukia or Eastern head regulator with 3 gated openings is on the right side of the Nile.

A classification of structural elements is shown in **Figure 8.1**. Typical levels and dimensions taken from historical drawings are given in **Table 8.1**.

### Table 8.1: Head Regulators - Typical Levels and Dimensions

|  | Western Head Regulator (Fouadia Canal) | Eastern Head Regulator (Faroukia Canal) |
|---|---|---|
| Number of Openings | 6 | 3 |
| Vertical dimensions (elevation m asl): |  |  |
| Top level of wall A, B, C (see **Figure 8.1**) | 72.30 | 72.30 |
| Top level of road and new exploratory boreholes | 71.20 | 71.25 |
| Top level of upstream piers | 69.50 | 69.50 |
| Bottom level of wall B (= top of closed gates) | 67.50 | 67.50 |
| U/s water level, Jan.99 | 65.00 | 65.00 |
| D/s water level, 09/01/99 | 64.00 | - |
| D/s water level, 10/01/99 | 63.00 | - |
| D/s water level, 13-14/01/99 | 61.10 | 62.90 |
| Top level of d/s piers | 65.30 | 65.30 |
| Top level of granite slab in bays | 60.50 | 61.25 |
| Top level of concrete slab (pier section) | 60.00 / 60.10 | 60.85 |
| Bottom level of concrete slab | 56.50 | 57.75 |

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## Table 8.1 (contd.): Head Regulators - Typical Levels and Dimensions

| | Western Head Regulator (Fouadia Canal) | Eastern Head Regulator (Faroukia Canal) |
|---|---|---|
| **Number of Openings** | 6 | 3 |
| **Horizontal dimensions (m):** | | |
| Height of one bay | 7.00 | 6.25 |
| Width of one bay | 6.00 | 6.00 |
| Height of arch over the bay | 1.20 | 1.20 |
| | 1.90 | 1.90 |
| Width of one pier | 2.0 | 2.0 |
| Length of one pier | 15.58 | 15.58 |
| Width of the road | 6.00 | 6.00 |
| Grooves for stoplog, width | 0.40 | 0.40 |
| Grooves for stoplog, depth | 0.30 | |
| Grooves for 2 gates, width | 1.55 | 1.55 |
| Grooves for 2 gates, depth | 0.36 | |

The two head regulators were designed by Wilson & Mitchell, Consulting Engineers, Westminster 5W (London/UK). The contract drawings are dated August 1926. The drawings referring to the existing structures are **Album Nos. 80** to **83**, see **Volume 4** of the Tender Design Report.

The gate openings have a height of 7.00 m (western) and 6.25 m (eastern), and a uniform width of 6.00 m. Each opening has two gates, which can be lifted and lowered by means of a gantry crane. The fabrication plates of the cranes show the year 1929.

Between the openings are piers with a length of 16.0 m and a thickness of 2 m (identical for both head regulators). In the western regulator, they are placed on a 4.0 m and in the eastern regulator on a 3.5 m, thick horizontal mass concrete slab, which includes at the top a layer of dressed granite paving blocks. The space between the piers can be separated by setting stoplogs on the up- and downstream end of the piers.

The foundation slab extends upstream and downstream of the pier structure with reduced thickness to a total length of 52.0 m at the western and 65.8 m at the eastern head regulator. There is no structure which limits energy dissipation to a certain length behind the gates.

The materials from which the two head regulators are constructed, are from top to bottom:

- Lime concrete filling under the roadway: It consists of sandy mortar with broken limestone (size 5 cm);

- Portland cement concrete for the arches of the roadway: The arches are made of concrete with a size 0 to 30 mm.

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- Limestone rubble masonry in cement mortar for the piers: The piers are made of a sandy mortar (sand up to 2 mm, sometimes up to 20 mm) with limestone size 15 cm to 20 cm (sometimes 5 to 40).

- Portland cement concrete for the base slab: The base slab is made of concrete with gravel and sand from 0 to 60 mm with a gap in size 2 to 15 mm.

Both, the abutment and the intermediate piers are made of rubble masonry blocks and stones, contained by a limestone (or granite) mosaic facing in the central part and limestone ashlar at the upstream and downstream ends.

There is a heavy traffic load on the eastern head regulator. At both structures no load limitations are indicated by sign boards or any other announcement. Several stairs up- and downstream of the structures allow an easy access to the water.

The following information was obtained from two MWRI engineers on site (which are there on duty since 12 and 20 years, respectively):

- Grouting of the piers and the abutments took place in 1987. On each pier, 3 vertical holes were drilled from the road area into the foundation slab (in total 21 on the western head regulator and 12 on the eastern head regulator). The length of these holes was between 11.5 and 12.0 m. The recovered cores are still available on site.

  The grouting was performed with normal cement and bentonite (2.5 % of cement content). The amount of water was 10 times the weight of cement at the beginning and 1 to 1 at the end. Pressure was between 1 and 2.5 bar. The main results from the final report on the drilling and grouting operation are listed in **Tables 8.2** and **8.3**.

- The scour protection at the end of the base slabs was refilled some years ago.

- All gates are still the original ones. Only the chains were replaced after years of operation. The material wears slowly due to small vibrations.

- The manpower operated cranes were upgraded with electric motors.

- Before High Aswan Dam (HAD) was built, the two head regulators were opened during the flood. Since operation of the HAD, the discharge in the two canals became limited to irrigation needs, and discharge control is performed by the upper gates only.

- The daily discharge in the western canal is at present some 70 m³/s in wintertime, with a maximum of 139 m³/s in summertime, whereas in the eastern canal the daily maximum is some 58 m³/s.

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## Table 8.2: Western Head Regulator - Cement Absorption from Grouting in 1987

| Borehole Location | | Absorption between Depths | | | | Total Absorption | | Core Recovery |
|---|---|---|---|---|---|---|---|---|
| | | 0 to 3 m | 3 to 6 m | 6 to 9 m | 9 to 11.6* m | Total | | |
| A 1 | upstream | 50 | 87.5 | 37.5 | 0 | 175 | 63% | 100% |
| A 1 | centre | 12.5 | 50 | 50 | 25 | 137.5 | 50% | 100% |
| A 1 | downstream | 100 | 100 | 50 | 0 | 250 | 90% | 100% |
| A 1 | | 162.5 | 237.5 | 137.5 | 25 | 562.5 | 203% | 100% |
| | | | | | | | | |
| 1 | upstream | 125 | 75 | 62.5 | 37.5 | 300 | 108% | 96% |
| 1 | centre | 100 | 150 | 100 | 25 | 375 | 135% | 96% |
| 1 | *downstream* | *75* | *50* | *125* | *25* | *275* | *99%* | *100%* |
| BG1 | | | | | | | | |
| 1 | | 300 | 275 | 287.5 | 87.5 | 950 | 342% | 97% |
| | | | | | | | | |
| 2 | upstream | 150 | 75 | 175 | 25 | 425 | 153% | 100% |
| 2 | centre | 50 | 25 | 25 | 37.5 | 137.5 | 50% | 97% |
| 2 | downstream | 125 | 175 | 150 | 62.5 | 512.5 | 185% | 96% |
| 2 | | 325 | 275 | 350 | 125 | 1075 | 387% | 98% |
| | | | | | | | | |
| 3 | upstream | 137.5 | 200 | 175 | 25 | 537.5 | 194% | 97% |
| 3 | centre | 37.5 | 175 | 175 | 75 | 462.5 | 167% | 100% |
| 3 | downstream | 112.5 | 37.5 | 225 | 75 | 450 | 162% | 100% |
| 3 | | 287.5 | 412.5 | 575 | 175 | 1450 | 522% | 99% |
| | | | | | | | | |
| 4 | *upstream* | *100* | *62.5* | *200* | *37.5* | *400* | *144%* | *97%* |
| BG2 | | | | | | | | |
| 4 | centre | 75 | 125 | 50 | 50 | 300 | 108% | 100% |
| 4 | downstream | 50 | 175 | 50 | 75 | 350 | 126% | 100% |
| 4 | | 225 | 362.5 | 300 | 162.5 | 1050 | 378% | 99% |
| | | | | | | | | |
| 5 | upstream | 50 | 350 | 87.5 | 25 | 512.5 | 185% | 100% |
| 5 | centre | 25 | 125 | 75 | 25 | 250 | 90% | 97% |
| 5 | downstream | 100 | 100 | 25 | 37.5 | 262.5 | 95% | 100% |
| 5 | | 125 | 225 | 100 | 62.5 | 1025 | 369% | 99% |
| | | | | | | | | |
| A 2 | upstream | 50 | 25 | 25 | 0 | 100 | 36% | 100% |
| A 2 | centre | 75 | 25 | 125 | 75 | 300 | 108% | 97% |
| A 2 | downstream | 25 | 62.5 | 50 | 12.5 | 150 | 54% | 100% |
| A 2 | | 150 | 112.5 | 200 | 87.5 | 550 | 198% | 99% |
| | | | | | | | | |
| **Medium per hole:** | | | | | | | | |
| | upstream | 82.8 | 109.4 | 95.3 | 18.8 | 306.3 | 110% | 99% |
| | centre | 46.9 | 84.4 | 75.0 | 39.1 | 245.3 | 88% | 98% |
| | downstream | 73.4 | 87.5 | 84.4 | 35.9 | 281.3 | 101% | 99% |
| | Medium | 67.7 | 93.8 | 84.9 | 31.3 | 277.6 | 100% | 99% |
| | | | | | | | | |
| | Total: | 1,422 | 1,969 | 1,783 | 656 | 5,830 | 2100% | |
| | | | | | | | | |
| | Total pier 1 to 5 | 1,263 | 1,550 | 1,613 | 613 | 5,550 | 1999% | |
| | kg / m³ | 4.7 | 5.7 | 6.0 | 2.7 | 5.4 | | |

- * sometimes only 11 or 11.5 m (medium: 11.5 m);
- 0: only 9 m borehole;
- BG 1/2: boreholes drilled in Jan.1999 ;
- distance between the holes on the pier: 3 m and 3 m;
- there were no dates or sequences of the grouting available;
- the kg / m³ were estimated under the assumption of a horizontal length of 9m in the pier (for 3 holes);
- loaction plan see **Figure 8.1**.

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## Table 8.3: Eastern Head Regulator - Cement Absorption from Grouting in 1987

| Borehole Location | | Absorption between Depths | | | | Total Absorption | | Core Recovery |
|---|---|---|---|---|---|---|---|---|
| | | 0 to 3 m | 3 to 6 m | 6 to 9 m | 9 to 11 m | Total | | |
| A 1 | upstream | 100 | 50 | 50 | 0 | 200 | 59% | 100% |
| A 1 | centre | 125 | 37.5 | 25 | 0 | 187.5 | 55% | 100% |
| A 1 | downstream | 137.5 | 25 | 37.5 | 0 | 200 | 59% | 100% |
| A 1 | | 362.5 | 112.5 | 112.5 | 0 | 587.5 | 174% | 100% |
| 1 | upstream | *300* | *300* | *150* | *0* | *750* | *222%* | *100%* |
| BG3 | | | | | | | | |
| 1 | centre | 75 | 75 | 50 | 0 | 200 | 59% | 100% |
| *1* | *downstream* | 250 | 50 | 250 | 0 | 550 | 162% | 97% |
| 1 | | 625 | 425 | 450 | 0 | 1500 | 443% | 99% |
| 2 | upstream | 250 | 250 | 50 | 0 | 550 | 162% | 100% |
| 2 | centre | 62.5 | 75 | 50 | 0 | 187.5 | 55% | 100% |
| 2 | downstream | 325 | 150 | 25 | 0 | 500 | 148% | 100% |
| 2 | | 637.5 | 475 | 125 | 0 | 1237.5 | 366% | 100% |
| A 2 | upstream | 225 | 75 | 50 | 0 | 350 | 103% | 100% |
| A 2 | centre | 75 | 37.5 | 25 | 0 | 137.5 | 41% | 100% |
| A 2 | downstream | 125 | 87.5 | 37.5 | 0 | 250 | 74% | 100% |
| A 2 | | 425 | 200 | 112.5 | 0 | 737.5 | 218% | 100% |
| **Medium per hole:** | | | | | | | | |
| | upstream | 218.8 | 168.8 | 75.0 | 0 | 462.5 | 137% | 100% |
| | centre | 84.4 | 56.3 | 37.5 | 0 | 178.1 | 53% | 100% |
| | downstream | 209.4 | 78.1 | 87.5 | 0 | 375.0 | 111% | 99% |
| Medium | | 170.8 | 101.0 | 66.7 | 0 | 338.5 | 100% | 100% |
| Total: | | 2,050 | 1,213 | 800 | 0 | 4,063 | 1200% | |
| Total pier 1 and 2 | | 1,263 | 900 | 575 | 0 | 2,738 | 809% | |
| | kg/m³ | 11.7 | 8.3 | 5.3 | - | 8.4 | | |

- BG 3: boreholes drilled in Jan.1999 ;
- distance between the holes on the pier: 3 m and 3 m;
- there were no dates or sequences of the grouting available;
- the kg / m³ were estimated under the assumption of a horizontal length of 9m in the pier (for 3 holes);
- loaction plan see **Figure 8.1**.

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## 8.2 Inspections of the Civil Works

### 8.2.1 Programme

In consideration of the important function of the head regulators and in view of the changes in the water management, the tender design phase includes a thorough investigation of the present conditions and performance of the head regulators. The objective of the investigation is to obtain the information which will permit decisions on rehabilitation and necessary modification of these structures.

The inspection of the two head regulators took place between 7 and 15 January 1999. Simultaneously, the drilling contractor Misr Raymond drilled exploratory boreholes and performed water pressure tests. The location of the boreholes (BW) is given on **Figure 8.2**.

### (1) Borehole Location Western Head Regulator

- BW-1 in pier No. 1, near the downstream parapet;
- BW-2 in pier No. 4, near the central parapet.

### (2) Borehole Location Eastern Head Regulator, all just downstream of the central parapet:

- BW-3 in pier No. 1;
- BW-4 in the arch between the piers;
- BW-5 in the arch between pier No. 2 and the right abutment;
- BW-6 in the arch between pier No. 1 and the left abutment.

The three deep boreholes BW-1, 2 and 3 were completed as piezometers. In piezometers BW-1 and BW-3, the filter screen was set in the foundation directly below the concrete slab, in BW-2 the screen was set about 7 m deeper in the foundation. In all cases, the seal was placed from about one metre above the filter screen.

Visual inspection of the two head regulator structures was done from the crest and from the downstream side. Access from downstream was possible after complete closure of the head regulators, which started on 10 January on the eastern and on 11 January on the western regulator. There remained more than 60 cm of water on the bottom slab of the western head regulator, and access was possible by foot. For the eastern head regulator, the water remained at a depth of 1.6 m and access was possible by boat only.

Because of the noise by the flowing water, it was not possible to assess the soundness of the piers by hammering. However, tests by Schmidthammer were performed. Since the eastern head regulator could only be accessed by boat, it was not possible to use the Schmidthammer in its downstream area.

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### 8.2.2 Findings and Observed Defects

A detailed description of the observations made during the inspection of the two head regulators is listed in tabular form in **Table 8.4** for the western head regulator and in **Table 8.5** for the eastern head regulator. A selection of photographs taken during inspection, showing general aspects of the structure and specific defects is given in **Appendix 8.1**.The structures are in an acceptable overall condition, and observed defects are:

### (1)  Cracking in Parapet Walls

The cracking in the parapet walls (walls C and B, see **Figure 8.1**) are due to temperature or traffic load (**photos 5, 6** and **17** of **Appendix 8.1**).

There is a continuous horizontal crack in the first joint under the black basalt road surface (3$^{rd}$ block from the top). It seems that this crack is caused by the exposure of the 5 cm thick road surface of black basalt to the sun radiation.

Several vertical cracks (normally very small) are distributed all over the length of the two walls. With one exception (eastern head regulator, wall B, pier 2) they do not penetrate through the limestone blocks.

### (2)  Cracking  of Joints of Roadway Arches

The cracking of the joints of the roadway arches are caused by movement in a joint of the concrete arch or by temperature (**photos 11** and **16** of **Appendix 8.1**).

The roadway arches are made of pre-cast concrete with in situ concrete backfill. The arch is formed by four segmental slabs with joint filling by mortar. There are five rows of segmental arches which form the entire arch. Cracks were observed within the joints only, but their origin is not clear. They do not seem to continue onto the parapet wall, see **Figure 8.1**.

### (3)  Weathering of Limestone Block Facing

Weathering of the limestone block facing occurred mainly in the range of the water surface changes. This can easily be observed on some of the limestone blocks when lowering the water table by some centimeters. **Photos 8** and **20** show worst cases which are not representative. Frequently, a few millimetres of limestone are washed out in the area of water surface fluctuations.

### (4)  Damaged Blocks

Some blocks have been damaged by traffic, mainly on the abutments (**photo 16**). Three ends of the walls along the two sides of the road are damaged on the eastern head regulator and one on the western regulator.

## Table 8.4: Western Head Regulator - Summary of Inspection Results

| | higher than elevation 65.00 m asl | | higher than elevation 65.00 m asl | | | | |
| | Wall A | Wall B | Wall C - road side | Wall C- downstream side | Arch | Pier on the left side | Pier on the right side |
|---|---|---|---|---|---|---|---|
| **Opening 1** | left upstream side: part of stone missing (max. 60 * 20 over depth 15 cm) | end block damaged, vertical cracks of 0.5 mm in joint | 3 vertical cracks of 0.4 to 0.6 mm in joint | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, cracks in the joints | old cracks with vertical stripes of material washed out | no visible damages |
| **Opening 2** | no visible damages | no visible damages, 3 vertical cracks of 0.25 mm in joint | vertical cracks of 1 mm in joint (between Opening 2 and 3) | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages | small damage on two stones under wall B |
| **Opening 3** | no visible damages | no visible damages, vertical cracks of 0.5 mm in joint (between opening 2 and 3) | no visible damages | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages | no visible damages |
| **Opening 4** | no visible damages | no visible damages, some vertical cracks of 0.2 - 0.6 mm in joint | vertical cracks of 1 mm in joint | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages | no visible damages |
| **Opening 5** | no visible damages | no visible damages, some vertical cracks of 0.2 - 0.6 mm in joint | no visible damages | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages | no visible damages |
| **Opening 6** | no visible damages | no visible damages, some vertical cracks of 0.2 - 0.6 mm in joint | no visible damages | horizontal crack 0.5 - 2 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages | small damage on one stone under wall B |
| **Downstream End of Piers** | | | | | | some small dissolving on the limestone at the water line | |
| **Rails** | no damages | 2 nuts are missing | | | | | |
| **Lighting** | | | damaged on left side | | | | |
| **Erosion** | At the end of the horizontal concrete slab (elevation 60.50 m asl) is a small hole with a depth of approx. 1.3 m (elevation 59.20 m asl). | | | | | | |

## Table 8.5: Eastern Head Regulator - Summary of Inspection Results

| | higher than elevation 65.00 m asl | | higher than elevation 65.00 m asl | | | | |
| --- | --- | --- | --- | --- | --- | --- | --- |
| | **Wall A** | **Wall B** | **Wall C - road side** | **Wall C- downstream side** | **Arch** | **Pier on the left side** | **Pier on the right side** |
| **Opening 1** | left upstream side: horizontal crack in joint (5 blocks from top), from stairs to upstream wall | some very small vertical cracks in joint | end block damaged by traffic, vertical crack in joint of 0.8 mm close to pier 1 | horizontal crack 0.5 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages, small dissolving of some limestone blocks at the water line | no visible damages, small dissolving of some limestone blocks at the water line |
| **Opening 2** | no visible damages | vertical crack of 0.8 m in joint and through one stone, close to pier 2 | some damages by traffic | horizontal crack in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, small cracks in the joints | no visible damages, small dissolving of some limestone blocks at the water line | no visible damages, small dissolving of some limestone blocks at the water line |
| **Opening 3** | no visible damages | end block damaged by traffic | end block severely damaged by traffic, 2 horizontal crack in joint of 0.5mm (on left and right side of opening) | horizontal crack 1 - 1.5 mm in joint on bottom of 3. block (1/2 block under the road) | 4x5 elements with joint filling, cracks in the joints | no visible damages, small dissolving of some limestone blocks at the water line | no visible damages, small dissolving of some limestone blocks at the water line |
| **DS and US end of piers** | | | | | | some small dissolving on the limestone at the water line | |
| **Rails** | no visible damages | no visible damages | | | | | |
| **Lighting** | | | no visible damages on the steel structure of the lightening | | | | |
| **Erosion** | At the end of the horizontal concrete slab (elevation 61.25 m asl) is a small hole with a depth of approx. 1m on the left (elevation 60.25 m asl )and 1.3 m on the right (elevation 60.00 m asl) side of the channel. | | | | | | |

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There are further small defects, e.g.:

- Some material has been washed out of the abutment wall in opening 1 of the western head regulator. There are several vertical stripes of soft material on this wall (**photos 12 and 13**). They are covered with the same green seaweed as shown on all other parts of the wall and seem to be old.

- Some joint filling is missing (**photo 21**).

## (5)     Schmidt-Hammer Tests

Several series of tests with the Schmidthammer were taken on the different materials of the surface of the two structures. The results are shown in detail in **Figures 8.6** and **8.7**. The main results in N/mm$^2$ (+/- 20%) are summarised in **Table 8.6**.

**Table 8.6: Results from Schmidt-Hammer Tests**

|  |  | Western Head Regulator | | Eastern Head Regulator |
|---|---|---|---|---|
|  |  | Upper Area | Water Area | Upper Area |
| Joint | medium | 44 | 31 | 43 |
|  | range | 25 to 60 | 12 to 50 | 30 to 50 |
| Shear Key | medium | 62 |  | 50 |
|  | range | 50 to 75 |  | 35 to 60 |
| Limestone | medium | 44 |  | 42 |
|  | range | 30 to 65 |  | 20 to 55 |
| Granite | medium |  | 67 |  |
|  | range |  | 40 to 75 |  |

There are only a few places with low or normal values. Most of the values are high or very high. It must be considered that the Schmidthammer is an instrument for normal concrete with a accuracy of approximately 20%. The surface must be plain, otherwise some material of the surface will be destroyed and completely wrong results will occur. On the joints and on the limestone it was necessary to search for suitable test locations. Therefore the results are not representative for all joints and limestone blocks. However, for the shear key (**photo 7**) they are considered representative.

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**(6)     Positive Findings**

- The quality of joint fillings was in generally good condition;

- there was no sign of differential settlements;

- no abrasion was observed (but note that the base slabs were not visible);

- there was only little scour of approx. 1 m depth at the end of the downstream concrete apron;

- there is practically no sign of material destruction caused by temperature changes.

**(7)     Conditions of Grouted Piers**

In the course of the remedial grouting works, carried out for the main barrage in 1986, also the abutments and piers of the two head regulators have been treated. At each pier and abutment of the head regulators, three grout holes have been drilled. All boreholes were located downstream of the gate slots on the bridge. One hole was located near the upstream parapet of the bridge, one in the centre and one near the downstream parapet. All holes ended inside the concrete base slab of the weir structure. Logs of these holes, with notes on water absorption and grout takes, are available. The cores taken from the grout holes are stored on site. The information on the water pressure tests, given on the logs, does not suffice for a full interpretation of these tests.

**(8)     Guide Wall Conditions**

Both left and right, on the upstream and downstream ends of the two head regulators, are vertical walls made of limestone blocks. These walls do not show any sign of deformation or surface damages except for an insignificant weathering on the right downstream wall of the eastern head regulator (**photo 20**). They are in a good overall shape.

The inclined surface protection following up- and downstream to the vertical walls are in a bad shape. **Photo 10** gives a representative view. On the other hand it must be considered, that all other parts of the canal are made without any slope protection.

### 8.2.3    Petrographic Composition of the Cores from New Drillings

As far as the petrographic composition of the aggregates is concerned, the following materials are distinguished:

1)   The asphalt pavement of the road is placed on ballast, set in weak mortar. The ballast is primarily obtained from crushed Esawia limestone. On the eastern head regulator, this zone had the highest grout takes and the grout shows also quite prominently in the drill cores. Obviously, the ballast had originally been left with a high, large scale porosity.

2)   The concrete of the arches has been prepared with crushed aggregate from red granite. Whereas the cores taken from the concrete on top of the piers were of fairly good quality, the cores taken from the centre of the arches were recovered in small fragments and, especially in the case of borehole BW-6 on the eastern head regulator, with uncommon low recovery. On the surfaces of these fragments, a discoloured film of material was observed with poor adhesion with the coarse aggregates. The texture of these surfaces is well distinguished from fresh breaks of the concrete. Therefore, these weak elements in the concrete possibly derive from poor preparation of the concrete in the arch elements.

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3) The centre part of the piers consists of rubble masonry, constructed with large fragments of Esawia limestone, placed in mortar. The Esawia limestone is a kind of travertine which has an elevated macro-porosity. Also, the mortar did not entirely fill the voids between the stones. Therefore, the masonry has a high porosity. The 1986 grouting works only partially sealed these voids. The observed grout averages 1 to 2% of the cores. The heavier slurries gave a grout stone of good quality. Part of the grout, on the other hand, must have been very dilute and left only a powdery residue in the voids (water/cement ratios of up to 10 by weight have been reported). Apparently, grouting had commenced with the excessively dilute slurries and, therefore, the bond between grout and masonry is frequently weak. Borehole BW-2 encountered large granite stones at the base of the pier (about 9.8 to 10.0 m depth). According to the existing drawings, these stones have been placed to form a key between concrete and masonry.

4) The base slab concrete of the western head regulator consists of concrete prepared with fluvial gravel, petrographically comprising chert, quartz, metavolcanics and volcanics. In both boreholes, sub-horizontal fissures were observed on the core. Along these fissures, the mortar is oxidised and somewhat weakened. Indications of mechanical damage or of alkali reaction with the chert were not found. Most probably, these fissures correspond to construction joints in the concrete. The oxidation along the fissures indicates circulation of water.

5) The base slab of the eastern head regulator was constructed mainly with crushed limestone aggregate (presumably taken from more solid horizons of the Esawia quarry) and a minor proportion of fluvial gravel (quartz, volcanics). A fissure at 13.45 m depth is coated with cement laitance and, therefore, is believed to represent a construction joint. Weathering at this joint is not as notable as on the cores of the western head regulator.

The 1986 grout holes have been drilled 0.5 to 0.6 m into the concrete base slab. But in none of the three new boreholes traces of grout were detected in the concrete core. Average specific grout takes in the lowest segment of the borehole averaged less than half of the specific takes in the upper 9 m of the piers of the western head regulator. In the eastern head regulator, the last 2 m between 9 and 11 m depth did not accept any grout at all.

## 8.3 Assessments

### 8.3.1 Structural Integrity of the Piers

During the head regulators' inspections, drilling of 3 vertical holes through the piers and 2 holes in the arches were undertaken, from which cores of good quality were recovered and in which water tests were performed.

The recovered cores reflect the material expected in accordance with the historic construction drawings. The materials from top to bottom are:

**(1) Lime Concrete Filling** (1.45 m)

- sandy mortar with broken limestone (size 5cm)

**(2) 4 to 1 Portland Cement Concrete** (arch, 0.7 m)

- good concrete, size 0 to 30 mm

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**(3)  Limestone Rubble Masonry in 2:1 Cement Mortar** (pier no. 9/8.15 m)

- Sandy mortar (sand up to 2 mm, sometimes up to 20 mm)

- with limestone size 15 to 20 cm (sometimes 5 to 40 cm)

**(4)  6 to 1 Portland Cement Concrete** (base slab, 3.5/3.1 m)

- Concrete 0 to 60 mm with a gap in size between 2 and 15 mm

The two concrete types in the arches and in the base slab seem to be of an adequate quality although the gradation of the aggregates is not as required to today's standard. The cores recovered from the base slab indicate that the concrete is sound and has no signs of cement leaching. This is also confirmed by the water pressure tests.

The limestone rubble masonry in the piers consists of limestone blocks embedded in mortar. The contact between mortar and blocks was found to be good, and only occasionally small gaps were seen between the two materials. These voids are limited in size. There are no signs that leaching has taken place within the rubble masonry, neither in the mortar, nor within the limestone blocks. These results are confirmed by the drilled cores.

The results of tests on the compressive strength and the unit weight of the core samples are shown in **Table 8.7** while their petrographic description is given in **Section 8.2.3**. Values are low, but with only small variance.

### Table 8.7: Borehole Samples - Compressive Strength and Dry Unit Weight

| Sample | Head Regulator | Remarks made in Jan. 99 | | Compressive Strength kg/cm$^2$ | Unit Weight Dry g/cm$^3$ |
|---|---|---|---|---|---|
| **Lime Concrete Filling:** | | | | | |
| 1-1 | West | | better | 81.1 | 2.15 |
| 2-1 | West | uniform, sandy mortar | representativ | 56.8 | 2.14 |
| | | **Lime Concrete Filling** | **Lowest Values** | **56.8** | **2.14** |
| | | | **Mean** | **69.0** | **2.15** |
| | | | | | |
| **4 to 1 Cement Concrete (Arch Concrete):** | | | | | |
| 1-2 | West | uniform | representativ | 218.3 | 2.37 |
| 1-3 | West | uniform | representativ | 209.2 | 2.36 |
| 2-2 | West | uniform | representativ | 177.0 | 2.37 |
| 3-1 | East | uniform, air traps | representativ to lower | 78.1 | 2.27 |
| 3-2 | East | uniform | representativ | 167.3 | 2.32 |
| 4-1 | East | uniform | representativ | 60.3 | 2.28 |
| 4-2 | East | uniform | representativ | 79.1 | 2.31 |
| 5-1 | East | uniform, air traps | better part of this core | 89.2 | 2.08 |
| | | **4 to 1 Cement Concrete** | **Lowest Values** | **60.3** | **2.08** |
| | | | **Mean** | **134.8** | **2.29** |
| | | | **Median** | **128.3** | **2.31** |
| | | | | | |
| **Limestone Rubble Masonry in 2: 1** | | | | | |
| 1-4 | West | only mortar with small stones | | 161.2 | 2.16 |
| 1-5 | West | limestone (normal) | | 125.3 | 2.29 |
| 1-6 | West | limestone (normal to bad) | | 68.9 | 2.12 |

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| Sample | Head Regulator | Remarks made in Jan. 99 | | Compressive Strength kg/cm$^2$ | Unit Weight Dry g/cm$^3$ |
|---|---|---|---|---|---|
| 1-7 | West | limestone ( hard with holes | | 104.8 | 2.24 |
| 1-8 | West | mix of mortar, limestone granite with inclined joint | low value expected | 112.8 | 2.22 |
| 2-3 | West | core diameter small | | 114.4 | 2.21 |
| 2-4 | West | mostly mortar | representative | 125.3 | 2.21 |
| 2-5 | West | limestone (normal to good) | | 108.2 | 2.30 |
| 2-6 | West | limestone (normal) | | **57.5** | 2.19 |
| 2-7 | West | mix of mortar, limestone, groutmaterial | | 67.0 | 2.17 |
| 2-8 | West | mix of mortar, limestone, groutmaterial with appr. horizontal joints | low value expected | 107.8 | 2.21 |
| 3-3 | East | mostly limestone | | 59.8 | 2.28 |
| 3-4 | East | only mortar | still wet after hours | 62.4 | 1.96 |
| 3-5 | East | mainly mortar, partly limestone, vertical joint | 60% mortar,40% limestone no inclined joint visible | **55.8** | 2.10 |
| 3-6 | East | 60% mortar, 40% limestone; no inclined joint visible | | 57.1 | 2.13 |
| 3-7 | East | mortar and limestone (big p.) | low value expected | 70.4 | 2.18 |
| 3-8 | East | limestone | low value expected | 67.9 | 2.13 |
| | | **Limestone Rubble Masonry in 2: 1** | **Lowest Values** | **55.8** | 1.96 |
| | | | **Mean** | 89.8 | **2.18** |
| | | | **Median** | 70.4 | 2.19 |
| **6 to 1 Portland Cement Concrete:** | | | | | |
| 1-9 | West | uniform | representative | 59.0 | 2.30 |
| 1-10 | West | uniform | representative | 63.4 | 2.25 |
| 1-11 | West | uniform | representative | 65.6 | 2.27 |
| 2-9 | West | uniform | representative | 61.4 | 2.28 |
| 2-10 | West | uniform | representative | 54.0 | 2.29 |
| 2-11 | West | uniform | representative | 52.6 | 2.29 |
| 3-9 | East | uniform | representative | 85.6 | 2.16 |
| 3-10 | East | uniform | representative | 60.6 | 2.16 |
| | | **6 to 1 Portland Cement Concrete** | **Lowest Values** | **52.6** | 2.16 |
| | | | **Mean** | **62.8** | **2.25** |
| | | | Median | 61.0 | 2.27 |

Water pressure tests were performed in the areas shown on **Table 8.1**, and the results are presented in **Figures 8.3, 8.4** and **8.5**. The tests were performed with a pressure between 1.3 and 2.4 bar at the test area, the length between packers was normally 2 m. In one of the tests, hydrofracturing presumably in the immediate vicinity of the borehole is indicated. In another test (BW-2, 6.5 to 8 m depth), a light erosion had started after 25 min of pumping. More frequently, the water absorption decreased with time, as the masonry filled with water.

Summary results in Lugeon (litre/min/m at 10 bar) are given in **Table 8.8**, in which the depths is measured in m below road deviation.

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## Table 8.8: Results from Water Pressure Tests (in Lugeon)

| Western Head Regulator | | | Eastern Head Regulator | |
|---|---|---|---|---|
| Depth in m | BW 1 | BW 2 | Depth in m | BW 3 |
| 3 to 5 | 35 | 3.5 | 3 to 5 | 8 |
| | | | 5 to 7 | 15 |
| 6.5 to 8.5 | 7.5 | 3.5 | 7 to 9 | 6.5 |
| 9.5 to 11.5 | 4 | 1.3 | 9 to 11 | 7.5 |
| 11.5 to 13.5 | 5 | 0.7 | 11 to 12.5 | 5 |

The results show that permeabilities in the lower halves of the piers and in the base slab are low, whereas higher values were measured in the upper halves. All boreholes passed through those parts of the piers which had already been grouted in 1986.

The consumption of cement grout from the 1986 grouting campaign is listed in **Tables 8.2** and **8.3**, the total cement consumption was 5,830 kg on the western and 4,063 kg on the eastern head regulator, which results in mean values of 5.4 and 8.4 kg/m$^3$. These values are low for this kind of structure. In the cores recovered from the 3 holes in the piers, typical traces of grout could be seen which did not fill the voids. The largest amount of grout was found in borehole no. 3.

Water absorptions in the masonry of the piers remain in a magnitude which should admit additional grouting, applying modern techniques with a suitable combination of injection pressures and grout takes. This would mean use of stable cement suspensions, preferably without bentonite, but with a super-plasticizer.

In the concrete the water absorption is not entirely negligible, confirming the presence of open construction joints, as already recognised on the core samples. On the other hand, the Lugeon units measured in the concrete, are at the limit for the application of normal cement slurries. Thus, if a sealing of the construction joints in the concrete apron would be required, special slurries would have to be used.

### 8.3.2   Foundation Soils

In-situ tests and laboratory analyses of soil samples from the boreholes show a notable difference in the foundation soils of the two head regulators, which are summarised in **Table 8.9**.

## Table 8.9: Characteristics of Foundation Soil

| Parameter | Fines <0.2 mm [%] | | | $D_{10}$ [mm] | | | CU | | | SPT-N | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Location | Aver. | Max. | Min. | Aver. | Max. | Min. | Aver. | Max. | Min. | Aver. | Max. | Min. |
| Eastern Head Regulator | 2.3 | 5.1 | 0.8 | 0.273 | 0.35 | 0.193 | 19.4 | 53.5 | 2.05 | >46 | >70 | 35 |
| Western Head Regulator | 6.2 | 21.4 | 0.8 | 0.143 | 0.31 | 0.006 | 3.2 | 6.5 | 2.03 | 25 | 34 | 8 |

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The soils in the foundation of the eastern head regulator are less uniform and generally of high density. In the upper levels of the foundation, borehole BW-3 encountered gravel (GW, GP) and well graded sand. This material may still be part of a fan of a lateral Wadi towards the river. At larger depth, the typical uniform sands of the Pleistocene Nile deposits prevail.

The foundation soils of the western head regulator resemble those encountered at the old barrage, i.e. mainly uniform fine to medium grained sand (SP). At larger depth, below elevation 51 m asl, also silty sand is intercalated (SM).

**Figure 8.8** displays the difference in the penetration resistance determined at the two sites (no corrections applied to the field data). At the western head regulator, the top of the uniform sand can have a low density with N-values less than 10 (as also found frequently in the foundation of the barrage). The mode of the N-values falls into the range of 20 to 30.

At the eastern head regulator, all N-values are larger than 30. This result may be to some extend influenced by the presence of large pebbles which could not be penetrated by the spoon. However, also the SP material at depth has a resistance exceeding 40.

For stability analyses, the following parameters shall be applied:

- foundation sand: $20 \text{ kN/m}^3$
- unit weight: $20 \text{ kN/m}^3$
- friction angle: $32°$

### 8.3.3 Conditions of Seepage and Uplift

According to the soil characteristics and the contents of fines, the foundation of the eastern head regulator should be more pervious than, for instance, the foundation of the old barrage. This agrees with observations made in relation with the pumping test on Dom island. The piezometer response during these tests indicated a recharge from the vicinity of the right abutment of the barrage, i. e. the vicinity of the Eastern Head Regulator.

The hydrographs for the three piezometers are shown in **Figure 8.9**. The quite high piezometric heads, measured in the centre of the structure, differ notably from the head distributions typically found in the foundation of barrages and gravity dams. The typical head distribution, if there are no grout or drainage curtains, almost linearly connects the upstream with the downstream water levels. At both head regulators, the piezometric head was measured notably above a hypothetical straight line. Several specific conditions can be made responsible for this situation:

- As determined by the pumping tests (see **Feasibility Study, Final Report**), the permeability of the soils in the Nile valley is highly anisotropic. For the upper sand strata, the horizontal permeability is at least 50 times higher than the vertical permeability.

- The headpond created by the existing barrage forms a recharge area which is much larger than the drainage area provided by the canal downstream of the regulators. Thus, the head established in the aquifer within the reach of the barrage will not be fully dissipated at the downstream end of the concrete apron of the head regulators. This applies especially here because the permeability anisotropy obstructs the upward drainage.

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For this reason, a design improvement with the aim to increase structural stability must take into account an unusual high uplift pressure under the base slab (see **Section 8.4 - Stability Analysis**).

## 8.4    Stability Analysis

### 8.4.1    General

Stability and stress analyses were carried out for both head regulators considering the future headpond level of 65.90 m asl, comparing stability against uplift, sliding including seismic forces, boiling and piping. The uplift forces were assumed in accordance with the observed high water tables under the foundation. Most of the reduction of the water pressure takes place at the end of the base slab (vertical permeability is 50 to 100 times smaller than the horizontal). Any transfer of forces by tension stresses should not be allowed.

In stability analyses, the lowest possible downstream water table which occurs after closure of all gates has to be considered. It must be expected that the downstream water levels will fall to:

- Western Head Regulator:    60.50 m asl,

- Eastern Head Regulator:    61.25 m asl.

The uplifting water pressure was considered linearly between the future head pond level, the observed piezometric heads in the boreholes on the head regulators and the lowest downstream water levels as given above. The resulting figures at the downstream end of the foundation are:

- Western Head Regulator:    63.12 m asl,

- Eastern Head Regulator:    63.52 m asl.

### 8.4.2    Loads and Load Cases

For stability analyses, the following unit weights were applied:

- arches and the piers:          $22 \text{ kN/m}^3$

- foundation concrete:          $23 \text{ kN/m}^3$

- new concrete:                     $24 \text{ kN/m}^3$

The friction angle for sliding analysis assuming a continuous crack between piers and concrete was assumed with $\alpha = 45°$.

The live load to be supported by each structure's platform shall remain for a 30 t vehicle as with the present status. An increase of live load to about 60 t would require a complete new super structure.

Impact of boats, floating wood pieces and the load from the gantry crane are causing very small stresses and can be neglected for re-calculation of stability.

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**Table 8.10** shows the load cases considered for the stability analysis of the structures against uplift, sliding and overturning.

### Table 8.10: Load Cases for Head Regulators

| Case | | Headpond Level m asl | Western Canal Tailwater Level m asl | Eastern Canal Tailwater Level m asl |
|---|---|---|---|---|
| Normal | | 65.90 | 64.00 – 65.50 | 64.00 - 65.50 |
| Unusual | | 65.90 | 60.50 | 61.25 |
| Exceptional | - Unusual + Earthquake | 65.90 | 60.50 | 61.25 |
| | - Emergency Releases | 67.05 | 64.00 | 64.00 |

The bases for present consideration on uplift conditions are few observations only. Based on these it has to be assumed that uplift pressure reduces only little in the horizontal reach below the structure itself. The major reduction occurs at the downstream end of the concrete base-slabs. This uplift pressure distribution is a conservative assumption of uplift conditions. To proof these assumptions more piezometers shall be installed and uplift pressure shall be measured during the rehabilitation of the base slabs. The base slabs are an important structural means to avoid boiling and piping of the foundation sand.

The present safety against piping and boiling (at the end of the base slab) is considered to be just acceptable when no scours occur. The existing small scours must be refilled in a safe way.

The admissible stresses for both head regulators are summarised in **Table 8.11**.

### Table 8.11: Admissible Stresses for Head Regulators

| Element | Typical Stress | Admissible Stress MN/m$^2$ |
|---|---|---|
| All parts | tensile stress | 0 |
| Concrete Arches | compressive stress | 1.0 ÷ 1.5 |
| Piers | compressive stress shearing stress | 1.2 0.08 |
| Foundation Concrete | compressive stress shearing stress | 0.5 0.1 |

Analyses were carried out first for the existing structure with the prevailing maximum headpond level of 65.10 m asl for normal (load case I) and unusual load cases (load case III), i.e. normal low tailwater level and minimum tailwater level which could occur during canal closure periods.

The impact of the headpond level increase to 65.90 m asl on the stability of the existing structures was analysed for the same tailwater conditions (load case II and load case IV).

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The results are summarised in **Tables 8.12** and **8.13**.

### Table 8.12: Western Head Regulator - Stability Analysis for Existing Structure

| Load Case | | | Piezometric Head | Factor of Safety against | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| no. | water level upstream | water level downstream | at downstream end | sliding | sliding with 10% earthquake | uplift 20 m from downstream end | uplift at downstream end | boiling | piping (Lane) |
| **Normal Load Case** | | | | | | | | | |
| I | 65.10 | 64.00 | 64.64 | 3.79 | 2.02 | 1.37 | 1.40 | 12.5 | 42.5 |
| II | 65.90 | 64.00 | 65.10 | 3.06 | 1.76 | 1.27 | 1.31 | 7.3 | 24.6 |
| **Unusual Load Case** | | | | | | | | | |
| III | 65.10 | 60.50 | 62.73 | 2.72 | 1.55 | **1.05** | 1.22 | 3.6 | 10.2 |
| IV | 65.90 | 60.50 | 63.12 | 2.17 | 1.33 | **0.96** | **1.13** | 3.1 | 8.7 |

### Table 8.13: Eastern Head Regulator - Stability Analysis for Existing Structure

| Load Case | | | Piezometric Head | Factor of Safety against | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| no. | water level upstream | water level downstream | at downstream end | sliding | sliding with 10% earthquake | uplift 21.5 m from downstream end | uplift 13 m from downstream end | uplift at downstream end | boiling | piping (Lane) |
| **Normal Load Case** | | | | | | | | | |
| I | 65.10 | 64.00 | 64.73 | 5.97 | 2.59 | 1.99 | 1.96 | 1.98 | 5.8 | 26.8 |
| II | 65.90 | 64.00 | 65.27 | 4.72 | 2.27 | 1.83 | 1.78 | 1.81 | 3.4 | 15.5 |
| **Unusual Load Case** | | | | | | | | | |
| III | 65.10 | 61.25 | 63.13 | 3.60 | 1.83 | 1.25 | **1.10** | 1.19 | 2.3 | 7.7 |
| IV | 65.90 | 61.25 | 63.52 | 2.82 | 1.56 | 1.15 | **0.99** | **1.08** | 1.9 | 6.3 |

Since safety of the base slabs against uplift did not proof to be sufficient, an additional concrete slab of 75 cm thickness was introduced for both head regulators in order to achieve sufficient safety.

The rehabilitated structures were analysed with the new headpond level of 65.90 m asl for normal tailwater conditions (load case V), for very low tailwater conditions during canal closure period (load case VI), and for extreme flood conditions (load case VII). These results are summarised in **Tables 8.14** and **8.15**.

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**Table 8.14: Western Head Regulator - Stability Analysis for Rehabilitated Structure (with new additional 75 cm Concrete Slab)**

| Load Case | | | Piezometric Head | Factor of Safety against | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| no. | water level upstream | water level downstream | at downstream end | sliding | sliding with 10% earthquake | uplift 20 m from downstream end | uplift at downstream end | boiling | piping (Lane) |
| **Normal Load Case** | | | | | | | | | |
| V | 65.90 | 64.00 | 65.10 | 3.13 | 1.79 | 1.50 | 1.56 | 7.3 | 24.6 |
| **Unusual Load Case** | | | | | | | | | |
| VI | 65.90 | 60.50 | 63.12 | 2.25 | 1.36 | 1.24 | 1.46 | 3.1 | 8.7 |
| **Exceptional Load Case** | | | | | | | | | |
| VII | 67.05 | 64.00 | 65.77 | 2.36 | 1.47 | 1.35 | 1.43 | 4.5 | 15.3 |

**Table 8.15: Eastern Head Regulator - Stability Analysis for Rehabilitated Structure (with new additional 75 cm Concrete Slab)**

| Load Case | | | Piezometric Head | Factor of Safety against | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| no. | water level upstream | water level downstream | at downstream end | sliding | sliding with 10% earthquake | uplift 21.5 m from downstream end | uplift 13 m from downstream end | uplift at downstream end | boiling | piping (Lane) |
| **Normal Load Case** | | | | | | | | | | |
| V | 65.90 | 64.00 | 65.27 | 4.78 | 2.29 | 2.07 | 2.06 | 2.10 | 3.4 | 15.5 |
| **Unusual Load Case** | | | | | | | | | | |
| VI | 65.90 | 61.25 | 63.52 | 2.88 | 1.58 | 1.44 | 1.36 | 1.48 | 1.9 | 6.3 |
| **Exceptional Load Case** | | | | | | | | | | |
| VII | 67.05 | 64.00 | 66.03 | 3.52 | 1.90 | 1.85 | 1.82 | 1.86 | 2.1 | 9.7 |

The calculation sheets for the individual load cases and different analyses are given as **Appendix 8.2**.

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### 8.4.3 Uplift

The safety factor against uplift is

$$\eta_{uplift} = \frac{\Sigma V_{downward}}{\Sigma P_{uplift}}$$

where:

$\eta_{uplift}$ = safety factor against uplift

$P_{uplift}$ = uplift forces; the resultant of these forces equals the gravity force of the volume of water displaced by a submerged structure

$V_{downward}$ = permanent vertical downward loads

Uplift is given through the remaining piezometric pressure below the structure on the volume of the slab. The piezometric line was established between the upstream head and the downstream head, i.e. between the observed water levels. However, the remarkable high piezometric heads observed in the foundation of the piers indicated that the major reduction in head is taken place immediately downstream of the slab. These extensions of the seepage path were considered for both head regulators for individually different ranges of values.

At the western head regulator the vertical length between the tailwater level and the bottom of the sheet pile wall at the end of the slab was weighted for the existing structure with a factor of 6.5. This represents a reduction of piezometric head of between 42% and 51% (depending on the load case) at the bottom of the sheet pile wall.

At the eastern head regulator the vertical length between the tailwater level and the bottom of the sheet pile wall at the end of the slab was weighted for the existing structure with a factor of 25. These represent a reduction in piezometric head of between 34% and 51% (depending on the load case) at the bottom of the sheet pile wall.

The weighted factors were calibrated according to the observed water levels in the foundation of the piers.

Analyses were first carried out for the existing structure with prevailing headpond level as it may be during the rehabilitation works. Thereafter analyses were carried out with the future raised headpond level, with and without new concrete slab.The analysed load cases are summarised in **Tables 8.12** to **8.15** above.

Most of the head reduction takes place at the end of the base slab (vertical permeability is 50 to 100 times smaller than the horizontal). The uplift forces on the downstream part of the base slab for both existing structures, eastern and western head regulators, are not acceptable. The factor of safety against uplift for the unusual load case is only between 0.96 and 1.13 (western) and between 0.99 and 1.08 (eastern), respectively. Resulting forces acting on this area are close to zero as shown in **Appendix 8.2**.

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After installation of an additional concrete slab with a thickness of 75 cm the factor of safety against uplift will become acceptable (**Appendix 8.2**). The required factors of safety (according to DIN 1045), as given in **Chapter 6, Table 6.5**, of 1.10 for normal and unusual load cases and 1.05 for exceptional load cases are met with sufficient margin. This is valid for the slightly higher requirements as defined by the Egyptian Building Code EC 196/1995 (1.20 for normal load case, 1.10 for unusual and exceptional load cases) too.

The analyses gave for the most critical conditions for the unusual load case factors of safety of between 1.24 and 1.46 (western) and between 1.36 and 1.48 (eastern), respectively. Not considered by this analysis was that the base slabs can not fail at an individual place but only as complete structural element. Therefore, the factor of safety for a structural element will be higher than the lowest calculated value.

### 8.4.4 Overturning and Sliding

Sliding stability was carried out for both head regulators under consideration of the weight of the piers. From the foundation slab only the 4 m thick part was taken into account. The friction angle of sand was taken as 32°.

The safety factor against sliding is

$$\eta_S = \frac{H_{f,base} + \Sigma H_{resist}}{\Sigma H_{actio}}$$

where:   $\eta_S$   = safety factor against sliding

$H_{f,base}$ = horizontal component of friction force acting on the base slab, where φ' is taken as specified by the geotechnical report

$H_{resist}$ = horizontal component of any other resisting force

$H_{actio}$ = horizontal component of any force acting on a structure

The results are summarised in **Tables 8.12** to **8.15** above. Details are given in **Appendix 8.2**. The main results of the statical computation for overturning and sliding revealed that they are no problem, for both head regulators, neither as existing nor as rehabilitated structures.

The factors of safety against sliding (without earthquake) are for the rehabilitated structures between 2.3 and 3.1 (western) and between 2.9 and 4.8 (eastern), respectively. These figures are well above the required figures of 1.50 (normal), 1.35 (unusual) and 1.20 (exceptional).

Similar results were obtained for the exceptional load case, i.e. unusual plus earthquake. The earthquake was taken as 0.1·g. Additional water pressure was considered according to WESTERGAARD. The obtained factors of safety are 1.36 (western) and 1.58 (eastern), both higher than 1.20.

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## 8.4.5 Boiling and Piping

The stability of the sand at the end of the slab was calculated against boiling considering the uplift forces and against piping according to LANE. Detailed results are given in **Appendix 8.2**.

Where as the factor of safety is in the order of 3, for the western head regulator, the safety against boiling (at the end of the base slab) at the eastern head regulator is 1.9. This is judged just acceptable when no erosion hole occurs.

The safety against piping is sufficiently given for the western head regulator with the lowest value of 8.7, which is higher than the required 5 to 6. However, at the eastern head regulator the factor of safety for the unusual load case is with 6.3 just above the required 5 to 6. Therefore, the existing small erosion holes must be refilled in a safe way. The base slab is very important to avoid boiling and piping of the sand.

## 8.4.6 Stresses

Stresses were analysed for the individual structural elements like concrete arches, piers and foundation slabs. The results are summarised in **Table 8.16**.

The stresses in piers and slabs are extremely low (even for earthquake, extreme high head pond level or cracked structure).

More critical are the stresses in the arches of the roadway compared to the bad results of the corresponding core samples. They were accepted taking into account, that the stress under a heavy truck increases the stress from dead load only by a factor of 2.0 to 2.5 and that the lowest tested sample still has a compressive strength 5 times higher than the calculated stresses.

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## Table 8.16: Stresses and Safety Factors

| | Western Head Regulator | Eastern Head Regulator | Remarks |
|---|---|---|---|
| **Concrete Arches** | | | |
| Lowest observed sample (compressive strength) | 17.7 N/mm² | 6.0 N/mm² | |
| of samples | 3 | 5 | |
| Truck of 60 t and dead load, compression stress in concrete | 1 N/mm² | 1 N/mm² | when considering only 50% of the concrete section (35 cm for distribution of concrete stress) |
| **Piers** | | | |
| Lowest observed sample (compressive strength) | 5.7 N/mm² | 5.6 N/mm² | |
| of samples | 11 | 6 | |
| Max. compressive stress on bottom of pier for | | | |
| - water tables: <br> u/s: 65.90 m asl and d/s on top of base slab | 0.32 N/mm² | 0.29 N/mm² | |
| - exceptional water table: <br> u/s: 67.20 m asl and d/s on top of base slab | 0.40 N/mm² | 0.33 N/mm² | |
| Horizontal force acting on bottom of the piers: | | | |
| - exceptional water table: <br> u/s: 67.20 m asl and d/s on top of base slab | | | |
| shearing stress: | 0.067 N/mm² | 0.053 N/mm² | |
| safety against sliding if completely cracked and 100% uplift in crack | 3.5 | >3.5 | |
| **Foundation slab** | | | |
| Lowest observed sample (compressive strength) | 5.3 N/mm² | 6.1 N/mm² | |
| of samples | 6 | 2 | |
| Max. compressive stress approximately same as in the piers. | | | |
| Possible tensile stresses on the upstream part of slab were not considered for the stability analysis (the whole upstream section was not considered). | | | |

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## 8.5    Reduction of Discharge Areas

After the implementation of the HAD, the full discharge capacity of the two head regulators for the periods of floods – which now have been eliminated – is no longer needed. The discharge capacity can orientate on the irrigation requirements and their extrapolation to the future.

The maximum discharge to be considered has been based on actual daily discharge records made available by the MWRI for the period 1990-1998. Calculations on the discharge capacity of one gate opening of each of the two head regulators are given in the **Appendix** to **Chapter 13** of **Volume 2, Hydro-mechanical Equipment - Rehabilitation of Head Regulators**. It is assumed that the head regulators can completely be operated by the "lower gates".

### (1)  Western Head Regulator

The observed maximum of daily discharge for the western head regulator is 164 m$^3$/s. Considering the potential increase of irrigated area by some 15,000 feddan[*] (4 %) the maximum required discharge capacity is 171 m$^3$/s. The corresponding d/s water level in the canal is 65.36 m asl. The minimum available head difference is 65.90 - 65.36 = 0.54 m.

To guarantee the required discharge capacity, the operation of the lower leafs of 3 gates at a height opening of 3.2 m is sufficient. 4 gates could be chosen for safety reasons in case of an unforeseen blockage of one gate. However, the MWRI instructed that all six gates should be maintained and rehabilitated for future operation.

### (2)  Eastern Head Regulator

The observed maximum daily discharge for the eastern head regulator is 56 m$^3$/s. Considering the potential increase of irrigated area by some 5,000 feddan (4 %) the maximum required discharge capacity is 59 m$^3$/s. The highest observed downstream waterlevel of 65.70 m asl has been taken as limitation. The corresponding head difference is 65.90 - 65.70 = 0.20 m.

To guarantee the required discharge capacity, the operation of the lower leafs of 2 gates at a height opening of 2.80 m is sufficient. However, 3 gates are chosen for safety reasons in case of an unforeseen blockage of one gate.

### (3)  Rehabilitation Concept

As all upper leafs of the gates originally used for flushing the canals during the yearly flood period are no more needed, they shall be replaced by concrete walls. As rehabilitation and adaptation of the lower leafs of the gate openings will cost approximately the same as new gates, it is foreseen to replace all 6 gates at the western head regulator and 3 gates at the eastern head regulator by new ones.

At the western head regulator the gates will be operated by hydraulic hoists. Since these gates have not to be moved simultaneously, only one hydraulic power unit is necessary. This hydraulic unit will be installed together with the required electrical equipment in a small building (2 m · 3 m) to be erected on the right upstream abutment of the structure.

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[*] 1 feddan = 4,200 m$^2$

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At the eastern head regulator the gates have to be operated during a few months only. Operation will be by the existing but refurbished gantry crane. The new gates will be of the same size and type as in the western head regulator. Thus it will be possible at any time to install the same hydraulic hoist system as it is now planned for the western head regulator, whenever the operation pattern changes and automation is needed.

## 8.6 Rehabilitation Works - Civil Construction

### 8.6.1 Western Head Regulator (Fouadia Canal)

#### (1) Drilling and Grouting

In the upstream end of piers 2 vertical holes on each of the 5 piers and 2 holes on each abutment, down to the sand of the foundation shall be drilled with subsequent grouting of all holes in ascending arrangement. The first step will be the grouting of possible cavities in the sand of the foundation.

In the downstream end of piers 1 vertical hole on each of the 5 piers and 2 holes on each abutment, down to the sand of foundation shall be drilled with subsequent grouting of all holes in ascending arrangement. The first step will be the grouting of possible cavities in the sand of the foundation.

#### (2) Piezometer Installation

In addition to the existing piezometres, two new piezometers shall be installed in the left and in the right downstream wall, which shall allow to measure the water pressure in the sand of the foundation.

#### (3) Filling of Erosion Holes at the End of the Base Slab

At the end of the base slab, there is a small scour, which must be filled with geotextile tubes or sacks filled with fresh concrete. If required, a geotextile will be placed first on top of the existing stones. The backfill shall be made up to a level between 60.50 and 61.00 m asl and over a length of 5 to 10 m.

#### (4) New Concrete Slab

A new concrete slab must be placed downstream of the piers, with approximate dimensions of 20 m · 48 m · 0.75 m. The slab has a top reinforcement layer and vertical anchor bars down to the old concrete.

#### (5) Wall Slabs Replacing Gate Sleeves

Openings Nos. 1 to 6 will be closed permanently only in the upper part, by reinforced concrete walls between the existing grooves. The dimension of these walls will be 3.75 m · 6.68 m · 0.55 m (h · l · t).

There will be connecting rebars to the old structure, and the 8 grooves (4 times 2) under these walls must be filled with reinforced concrete (3.25 m · 0.34 m · 0.55 m).

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## (6) Small Control Building

A small building (one room with dimensions: 2.0 m · 3.0 m · 2.5 m) shall be erected on the upstream right abutment (for hydraulic unit and control system of the gates) complete including small power and lighting.

## (7) Kerb Stones on Both Sides of the Head Regulator's Roadway

On the left and right side of the roadway kerb stones of 30 cm · 20 cm (width · height) must be set along the whole length (50 m). There will be some vertical anchor bars and a light reinforcement. Start and end of these kerb stones must be marked with reflecting plates.

## (8) Repair of Stone Facing and Setting of Seals

Some weathered limestone blocks in the area of headpond surface variation must be repaired. The stones must be washed with high pressure water. Special mortar for repair with the colour of the stone and an effective bonding agent must be applied.

## (9) Repair of the Inclined Wall Downstream of the Head Regulator

The inclined walls on the left and right side of the canal (between the normal canal section and the head regulator vertical walls) are in poor condition. They must be repaired by means of local stones, mortar and concrete (for filling of small holes).

### 8.6.2 Eastern Head Regulator (Faroukia Canal)

## (1) Drilling and Grouting

In the upstream end of piers 2 vertical holes on each of the 2 piers and 2 holes on each abutment, down to the sand of the foundation shall be drilled with subsequent grouting in all holes in ascending arrangement. The first step will be the grouting of possible cavities in the sand of the foundation.

In the downstream end of piers 1 vertical hole on each of the 2 piers and 2 holes on each abutment, down to the sand of the foundation shall be drilled with subsequent grouting in ascending arrangement. The first step will be the grouting of possible cavities in the sand of the foundation.

## (2) Piezometers Installation

In addition to the existing piezometers, two new piezometers shall be installed in the left and in the right downstream wall, which shall allow measuring the water pressure in the sand of the foundation.

## (3) Filling of Erosion Holes at the End of the Base Slab

At the end of the base slab, there is a small erosion scour which must be filled with geotextile tubes or sacks filled with fresh concrete. If required, a geotextile will be placed first on top of the existing stones. The backfill shall be made up to a level between 61.25 and 61.75 m asl and over a length of 5 to 10 m.

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### (4) New Concrete Slab

A new concrete slab must be placed downstream of the piers, with approximate dimensions of 20 m · 24 m · 0.75 m. The slab has a top reinforcement layer and vertical anchor bars down to the old concrete.

### (5) Wall Slabs Replacing Upper Gate Leaves

All 3 openings will be closed in the upper part by reinforced concrete walls between the existing grooves. The dimension of these walls will be 3.75 m · 6.68 m · 0.55 m (h · l · t).

There will be rebars connecting to the old structure, and the 6 grooves (3 times 2) under these walls must be filled with reinforced concrete (3.25 m · 0.34 m · 0.55 m).

### (6) Kerb Stones on Both Sides of the Head Regulators Roadway

On the left and right side of the roadway kerb stones of 30 cm · 20 cm (width · height) must be set along the whole length (25 m). There will be some vertical anchor bars and a light reinforcement. Start and end of the kerb stones must be marked with reflecting plates.

### (7) Repair of Stone Facing and Setting of Seals

Some weathered limestone blocks in the area of headpond surface variation must be repaired. The stones must be washed with high pressure water. Special mortar for repair with the colour of the stone and an effective bonding agent must be applied.

## 8.6.3 Traffic Safety Measures

Both head regulators have a function as bridge over the two main irrigation canals for public traffic. The eastern head regulator is presently used by heavy traffic of the national road, but this situation must change with rehabilitation. A new bridge for the east bank national road is planned to span the eastern canal. The following remarks have to be considered within the design of the rehabilitated structures:

- There is no marking of the dangerous ends of the side walls, limiting the road width on either end of structure. Reflecting plates should be installed to reduce the damages caused by traffic.
- Safety for people shall be achieved by adding railings on the structure and the abutments.

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### 8.6.4 Working Sequence for Improvement of D/S Apron

The following working sequence should be followed, whilst uplift safety conditions are monitored:

1) Drilling and grouting of the piers.

2) Installation of new piezometers.

3) Preparatory works.

4) Complete closure of the gates (If all gates were not originally opened, they must be opened regularly for several hours of flushing before being closed).

5) Filling of the erosion hole (under water) at the end of the base slab with textile bags filled with fresh concrete. The bags must be placed by divers up to the following elevation:

   - Western head regulator: 60.50 to 61.00 m asl (elevation of present base slab or a little higher).

   - Eastern head regulator: 61.25 to 61.75 m asl (elevation of present base slab or a little higher).

6) 3 to 4 days after complete closure of the gates, the following final downstream water levels are to be expected:

   - Western head regulator: 61.20 m asl (water depth over the base slab approximately 0.70 m).

   - Eastern head regulator: 63.00 m asl (water depth over the base slab approximately 1.75 m).

7) Installation (under water) of a 45 cm high formwork (or textile bags) close to the end of the piers and at the end of the slab.

8) 40 to 50 cm concrete filling (under water) between the formwork and the side walls and installation/finishing of a small temporary "dam" at the downstream end of the base slab.

9) Dewatering the base slab down to the top level of the original slab.

10) Drilling of holes for the vertical anchor reinforcement (each 1.6 m and 0.4 m at the end lines).

11) Concreting up to the final thickness of 75 cm.

12) Removal of the small temporary "dam".

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# Figures

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## Western Head Regulator (Fouadia)

DS or D (downstream)

Opening No:  1   2   3   4   5   6

Wall C

Street    A1 (left)   Pier 1   Pier 2   Pier 3   Pier 4   Pier 5   A2 (right)

Wall B

Opening for Gates

Wall A

US or U (upstream)

## Eastern Head Regulator (Faroukia)

DS or D (downstream)

Opening No:  1   2   3

Wall C

Street    A1 (left)   Pier 1   Pier 2   A2 (right)

Wall B

Opening for Gates

Wall A

US (upstream)

**Figure 8.1: Head Regulators - Schematic Structural Arrangement**

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## FOUADIA CANAL (WEST) HEAD REGULATORS
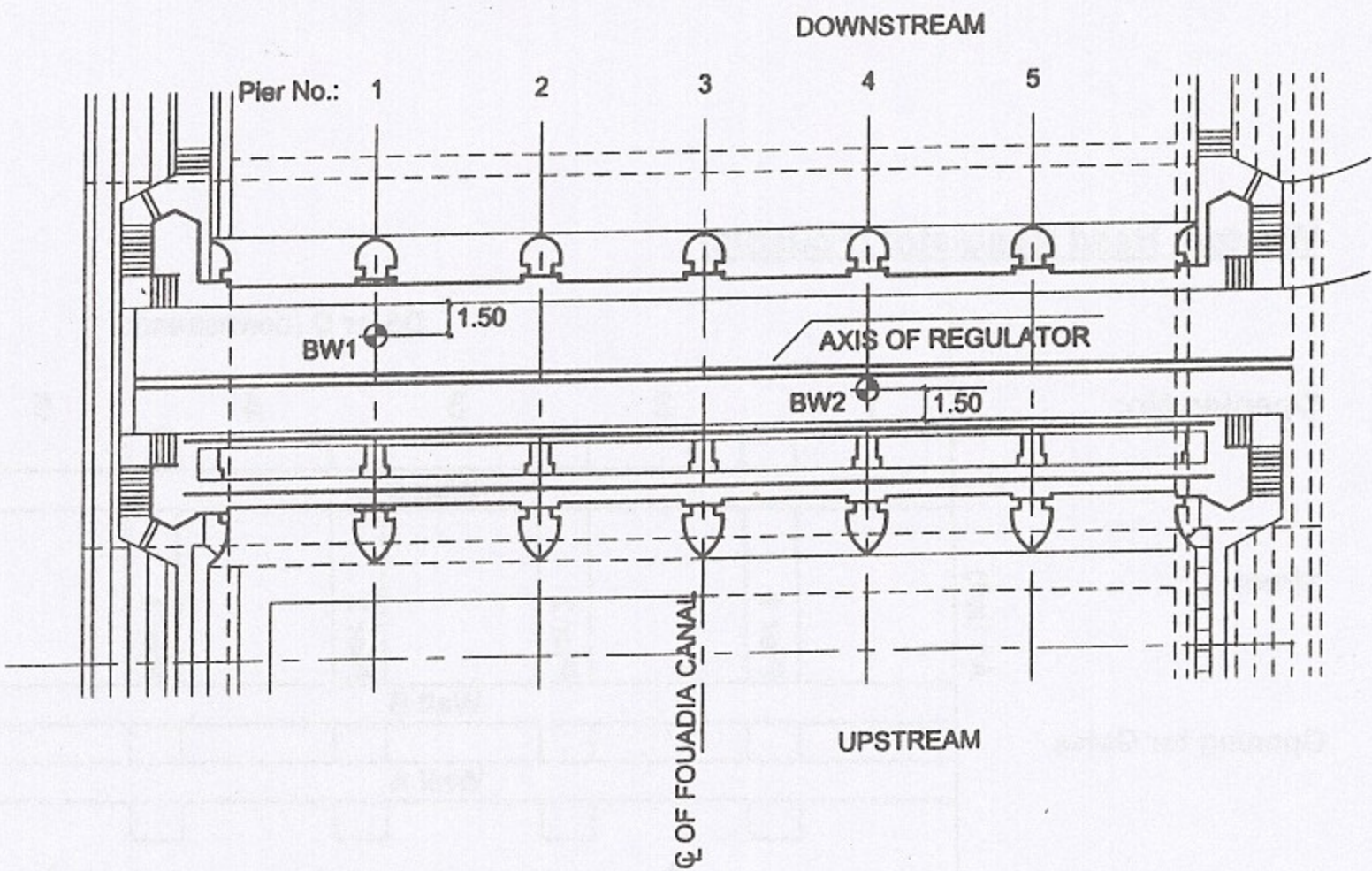


THIS DRAWING IS BASED ON GENERAL
DRAWINGS AS EXECUTED SCALE 1 : 200
PREPARED BY S. OF E. 1931 (31/600)
NAGA-HAMMADI BARRAGE DRG. NO. 838
REPORT DRG. NO. 41

## FAROUKIA CANAL (EAST) HEAD REGULATOR
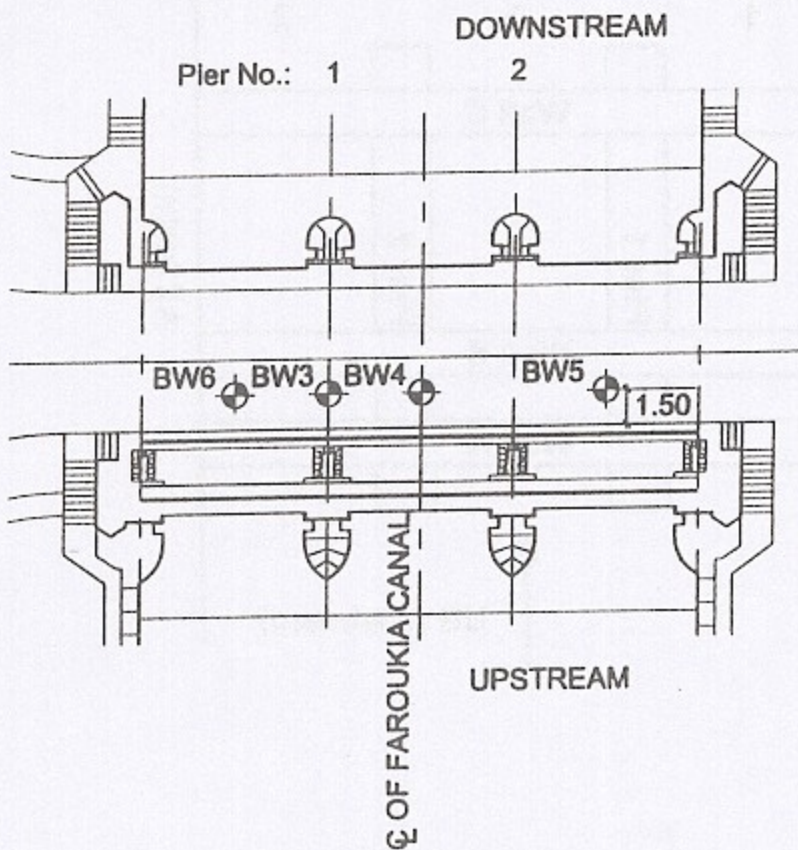


THIS DRAWING IS BASED ON GENERAL
DRAWINGS AS EXECUTED SCALE 1 : 175
PREPARED BY S. OF E. 1931 (31/600)
NAGA-HAMMADI BARRAGE DRG. NO. 819
REPORT DRG. NO. 43

SCALE

0   05   10   15   20   25   30 m

(ORIG. SCALE : 1/500)
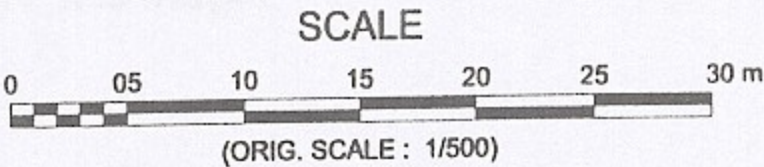
## Figure 8.2: Head Regulators-Location of Boreholes

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Figure 8.3a: Borehole BW 1



Figure 8.3b: Borehole BW 1 from 9.5 to 11.5 m
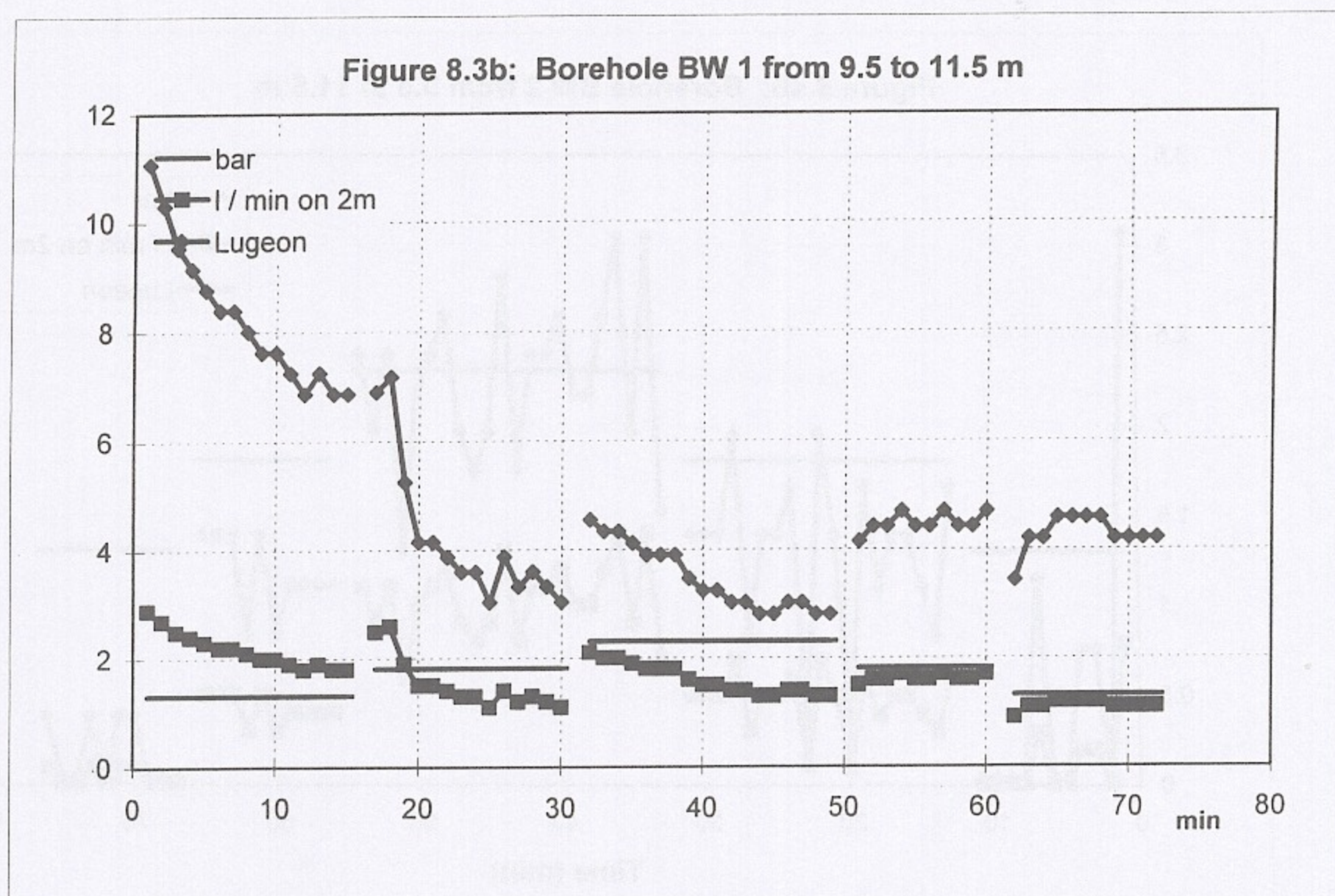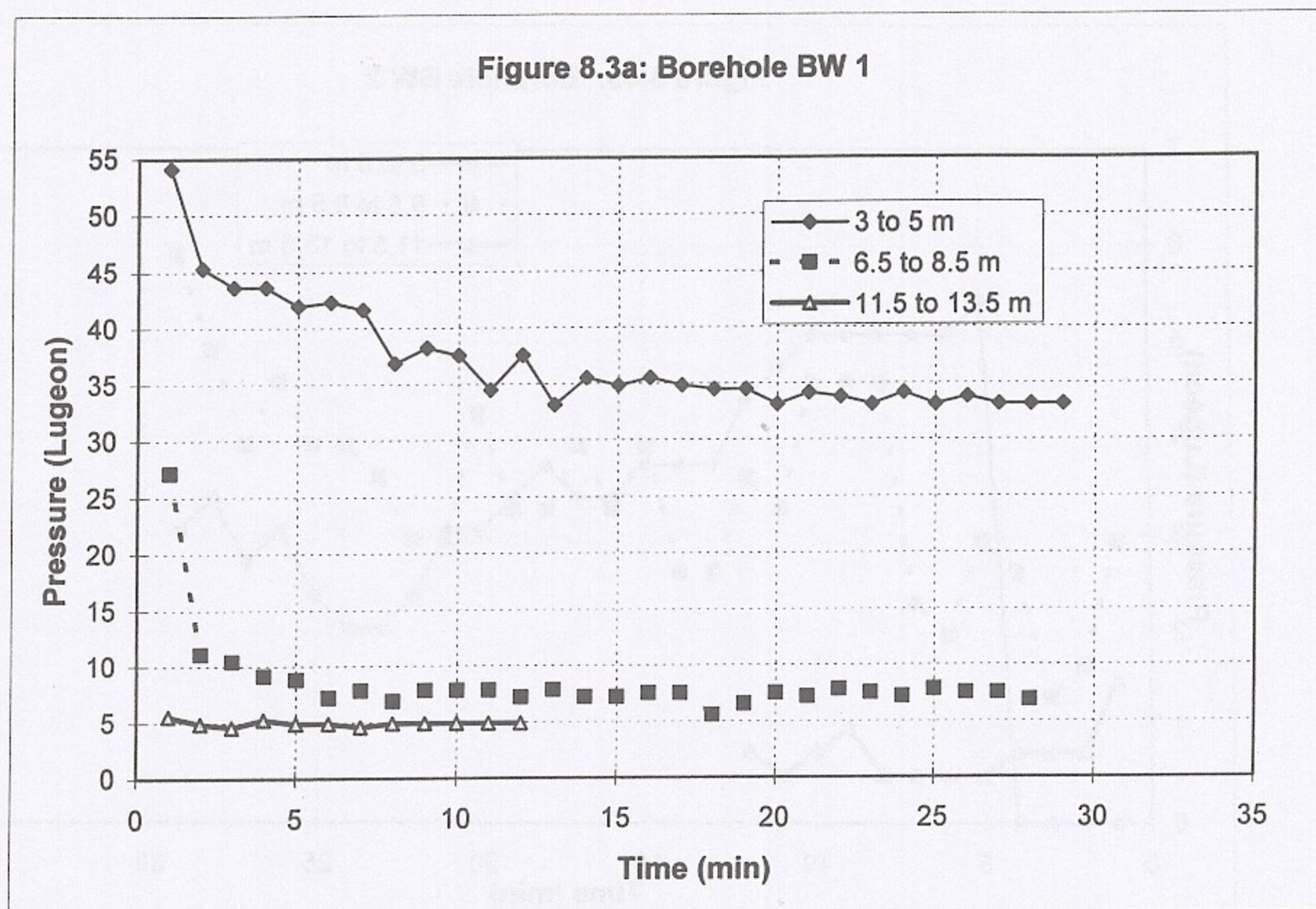
Figure 8.3: Western Head Regulator BW1 - Results of Water Pressure Tests

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Figure 8.4a: Borehole BW 2



Figure 8.4b: Borehole BW 2 from 9.5 to 11.5 m
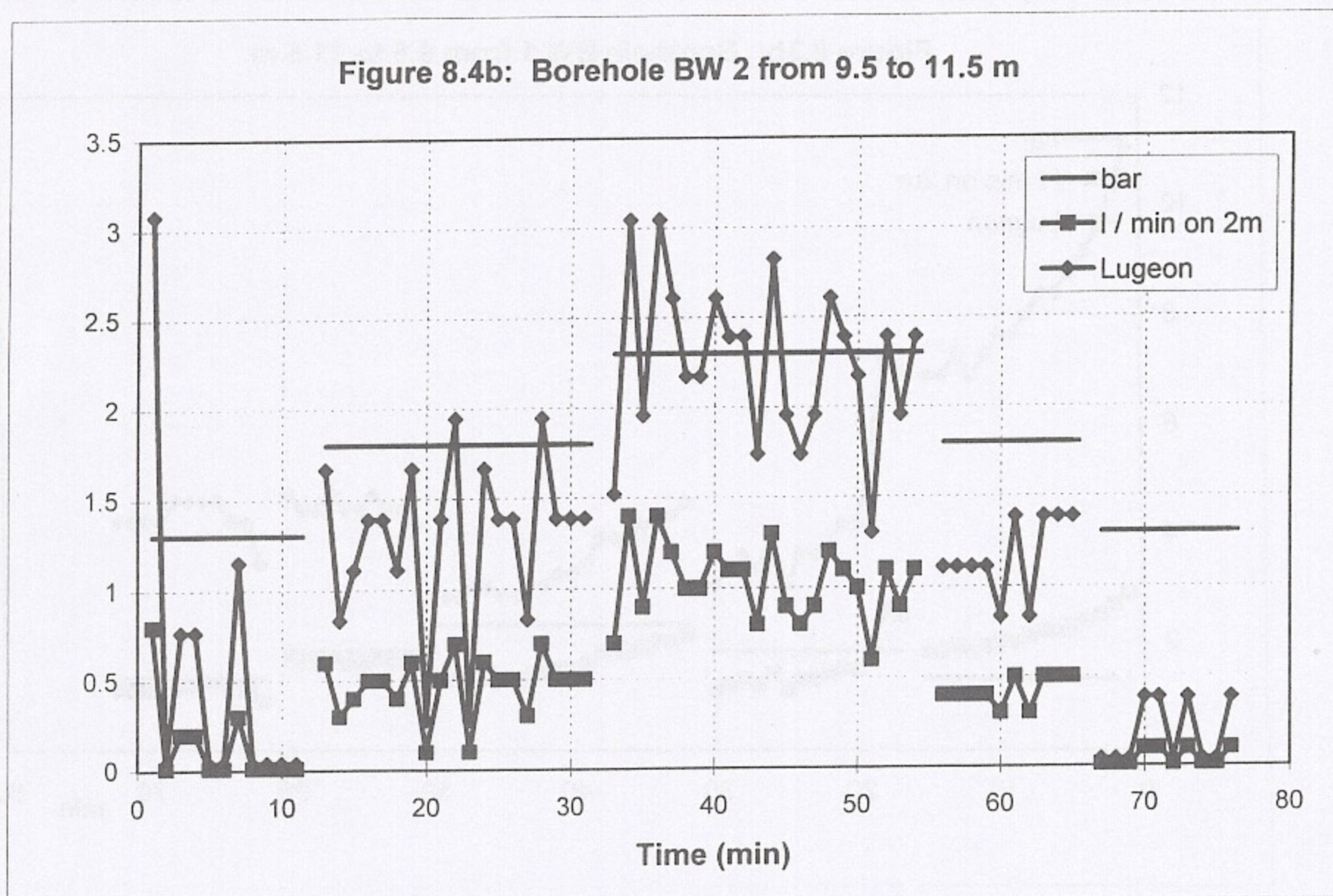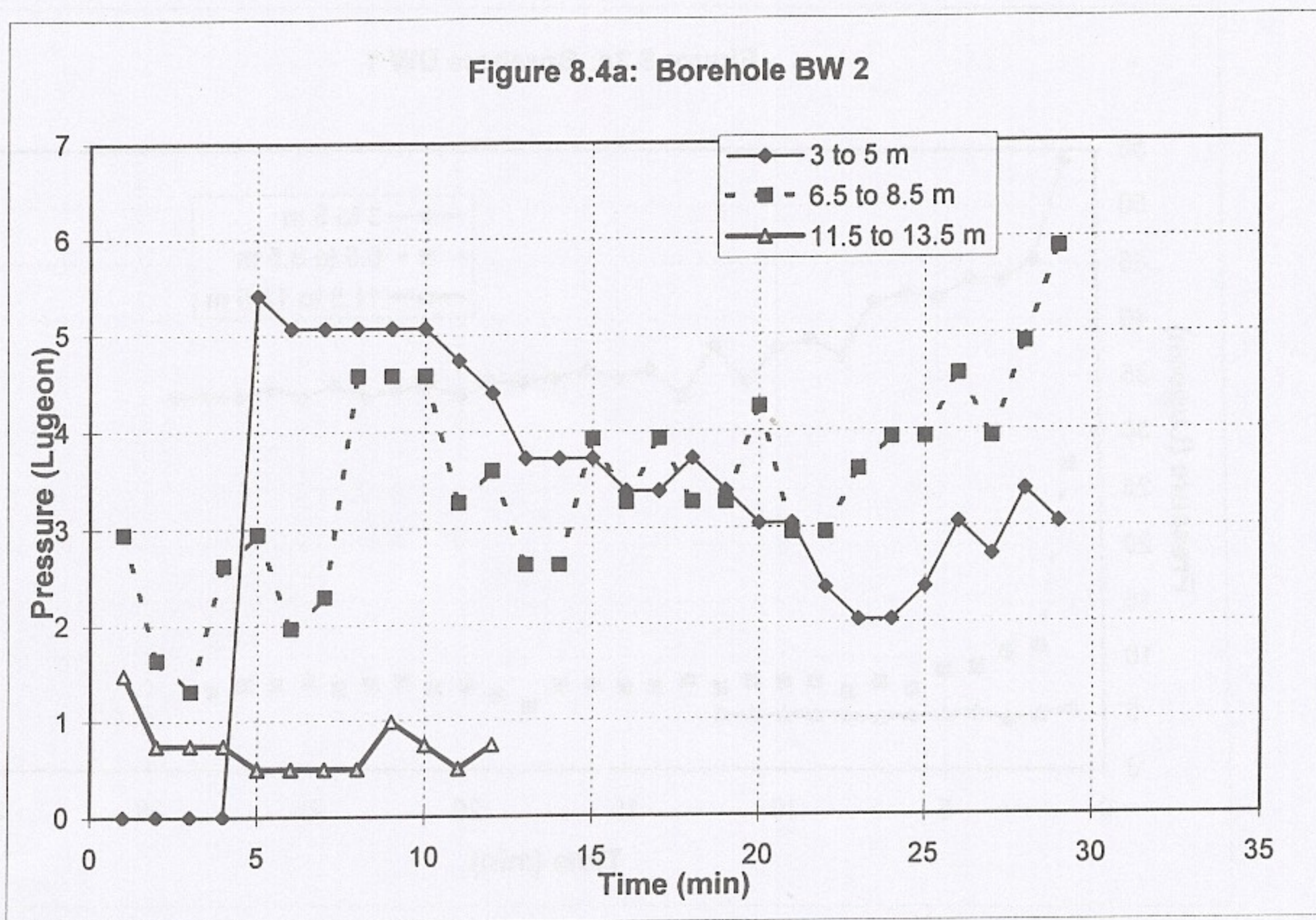
Figure 8.4: Western Head Regulator BW2 - Results of Water Pressure Tests

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**Figure 8.5a: Borehole BW 3**



**Figure 8.5b: Borehole BW 3 from 9.5 to 11.5 m**
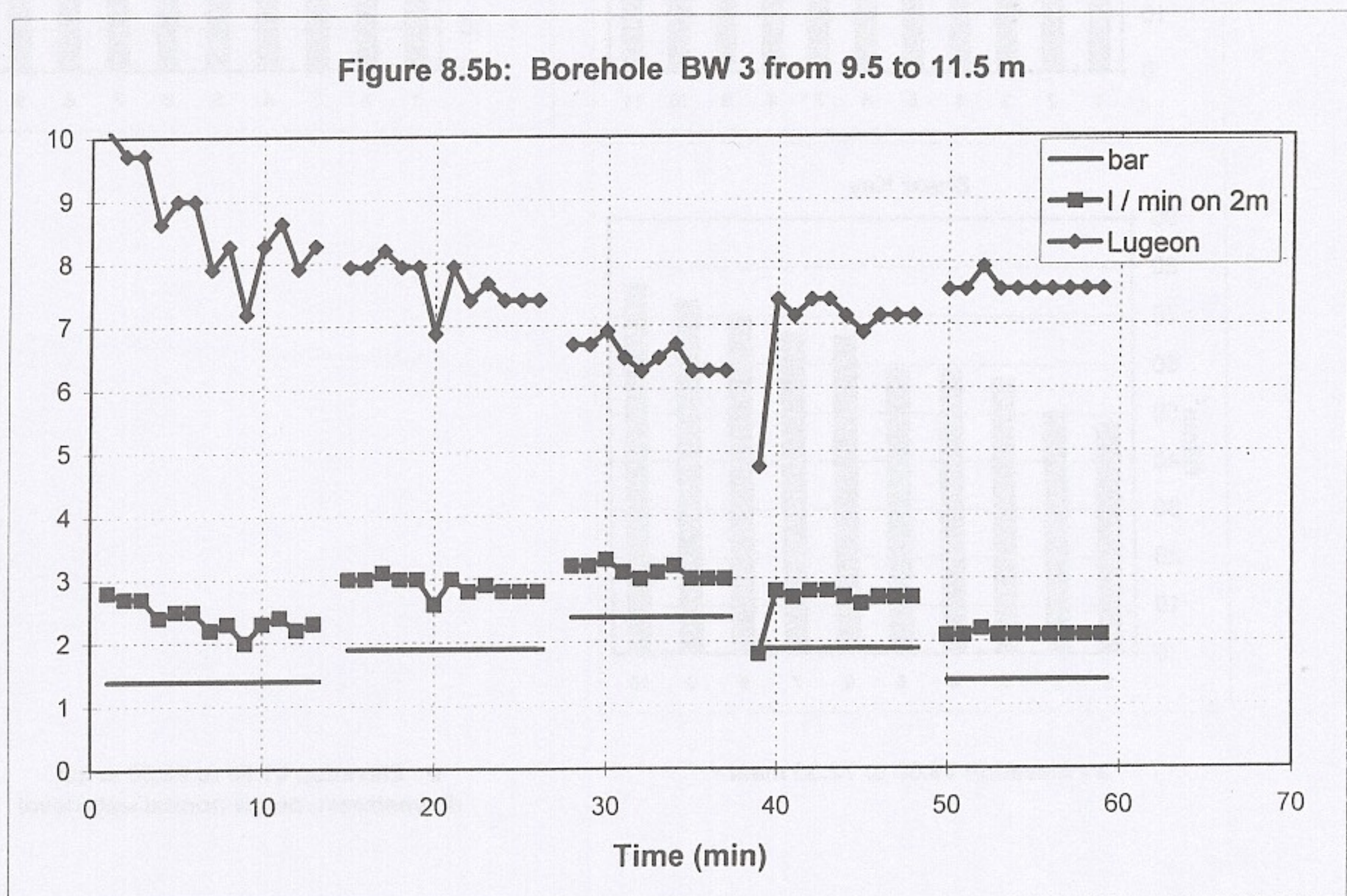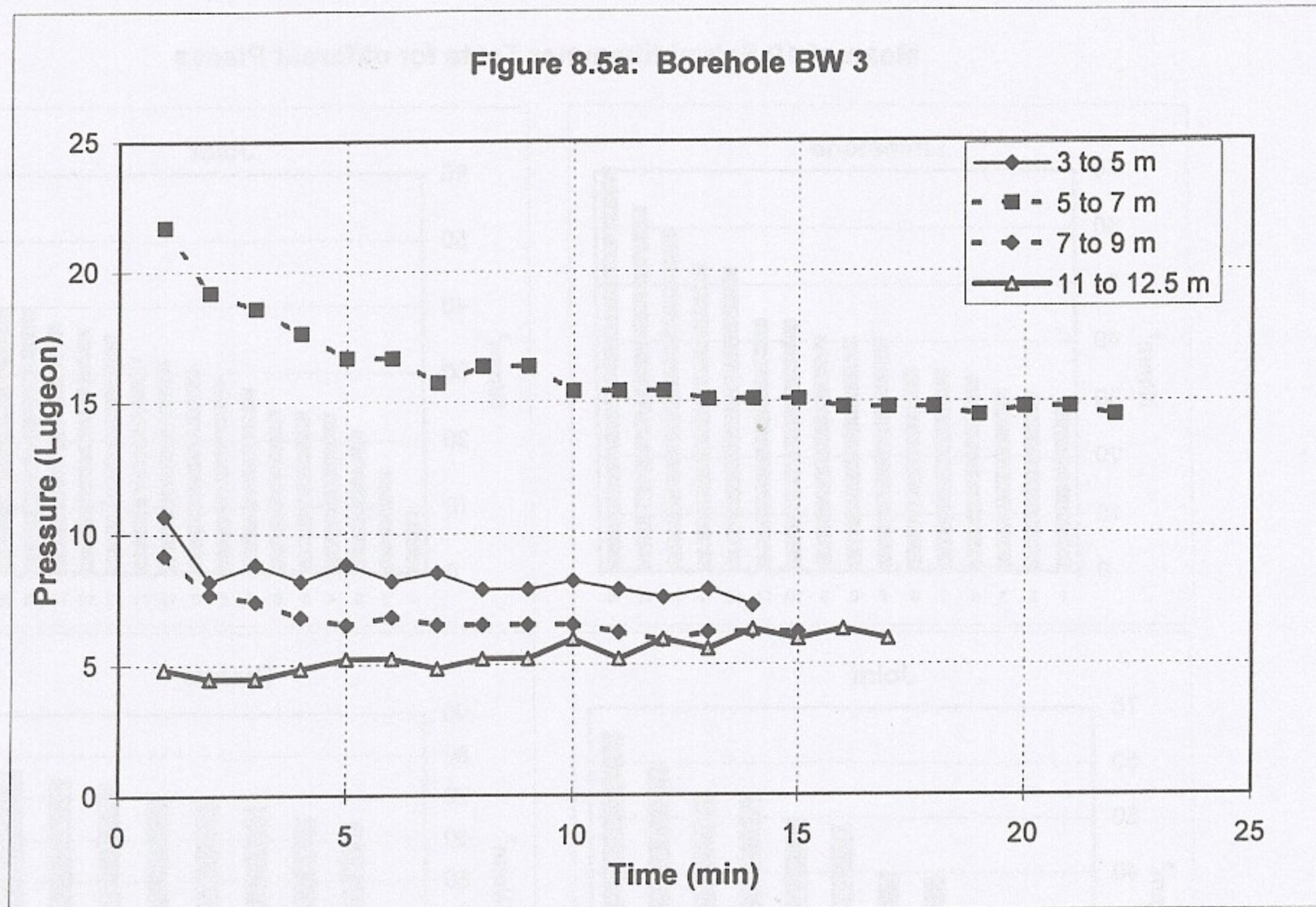


**Figure 8.5: Eastern Head Regulator BW3 - Results of Water Pressure Tests**

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**Mean of 10 Schmidthammer Tests for different Places**
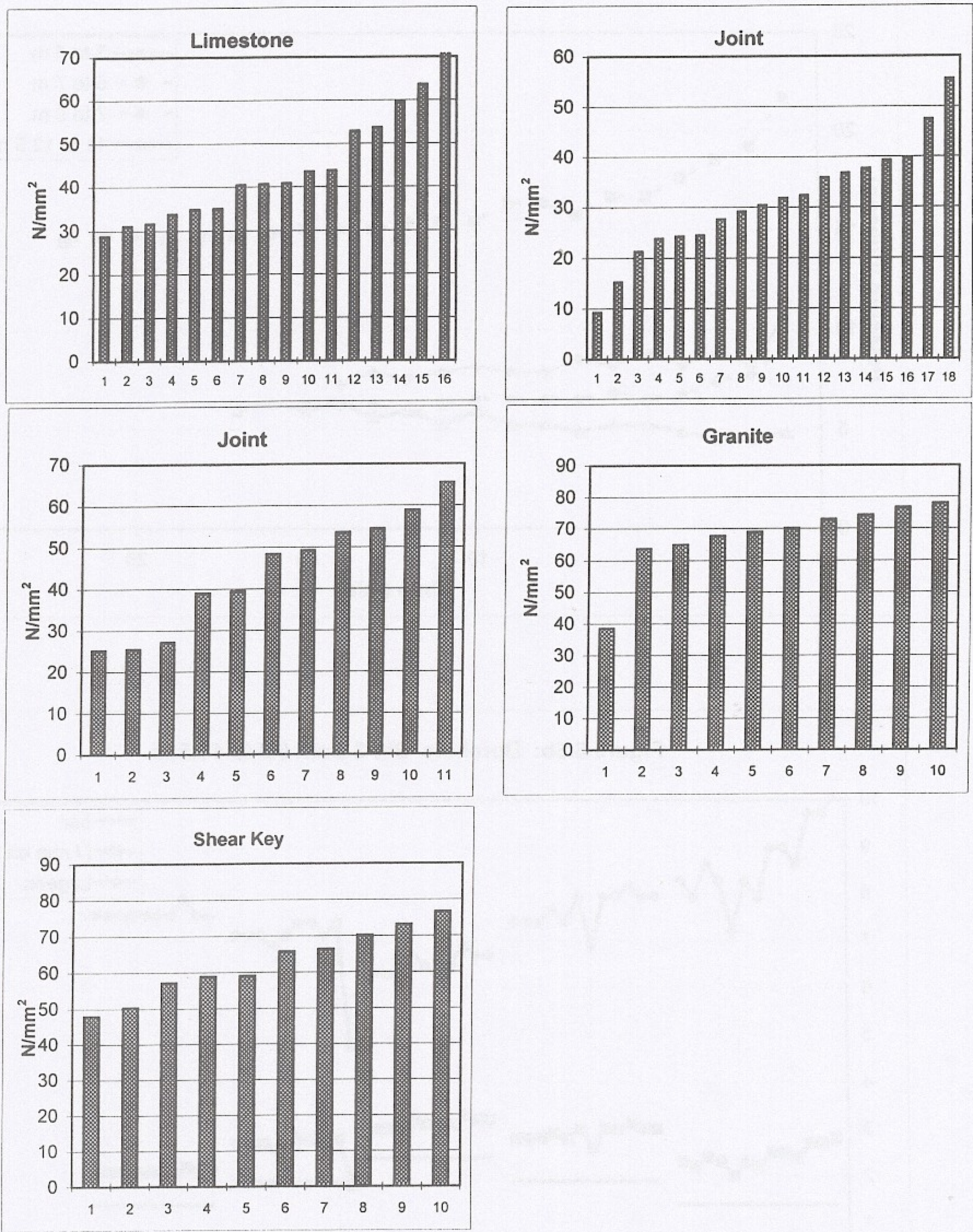


a - Elevation 69.00 to 72.30 masl

b - Elevation 61.50 to 62.50 m asl
downstream, below normal waterlevel

**Figure 8.6: Western Head Regulator - Schmidthammer Tests**

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## Mean of 10 Schmidthammer Tests for different Places
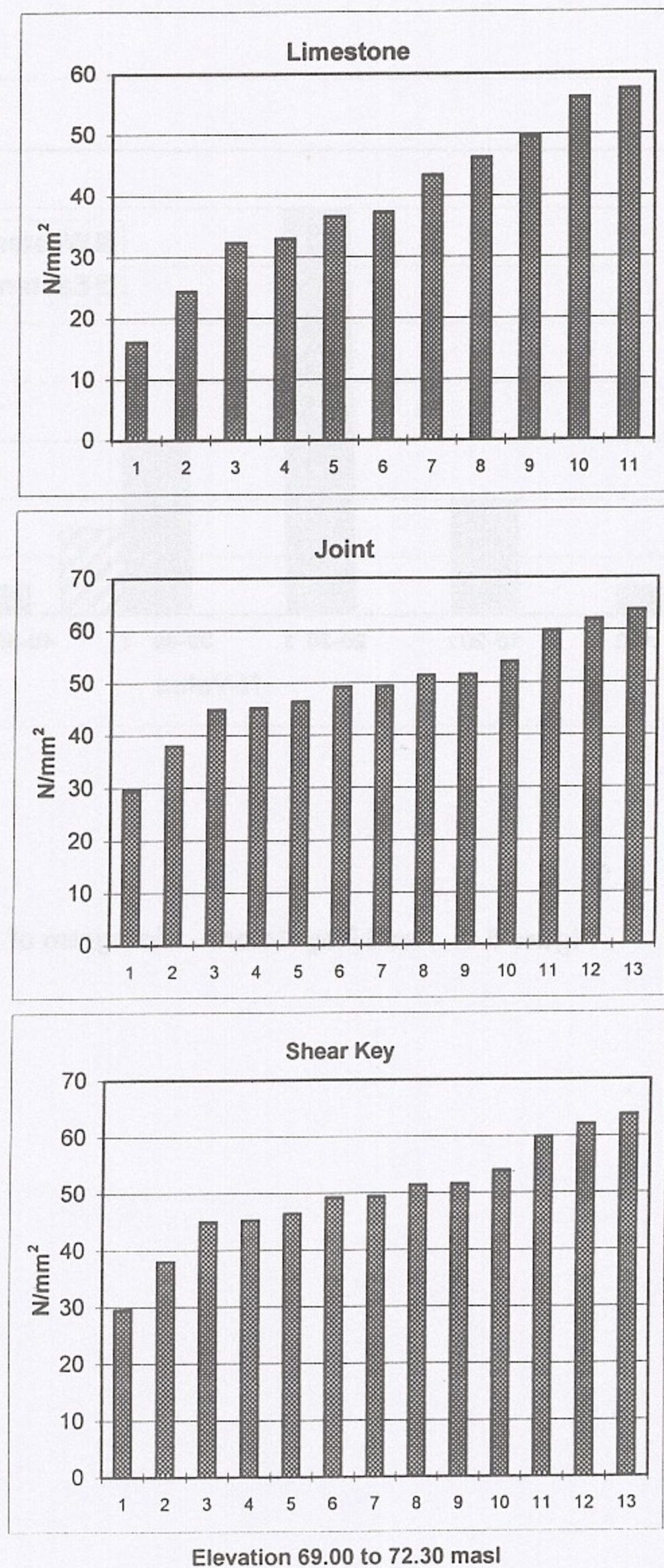


Figure 8.7: Eastern Head Regulator - Schmidthammer Tests

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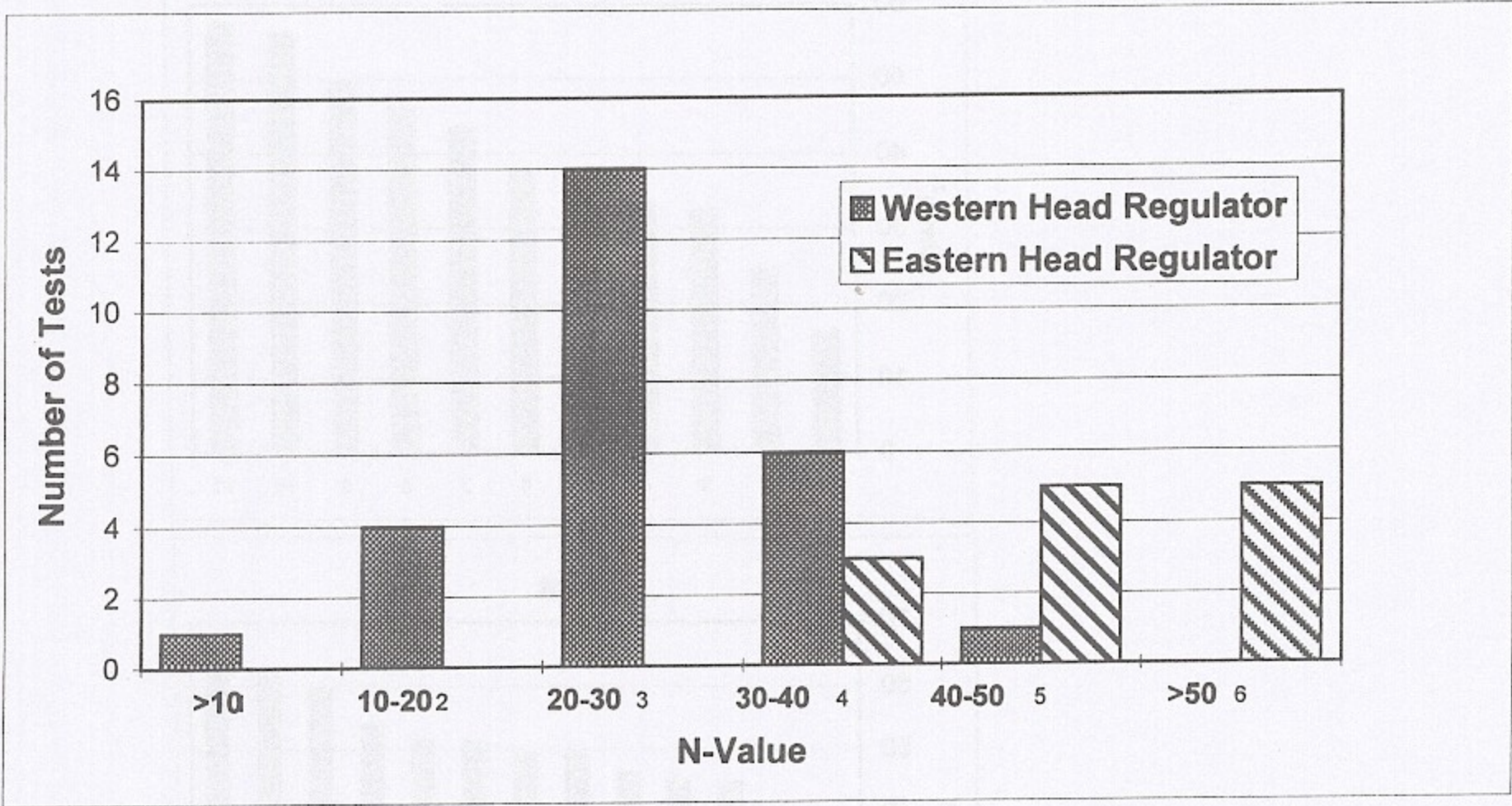
Figure 8.8: Head Regulators - Histogram of SPT-N Values

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## Observed Water Levels

Drawdown of Eastern Canal

Drawdown of Western Canal

**Date (1999)**

| | |
|---|---|
| ■ BW 1 Western Head Regulator | ◆ Headpond |
| ◇ BW 2 Western Head Regulator | ■ Western Canal |
| ✳ BW 3 Eastern Head Regulator | ✕ Eastern Canal |
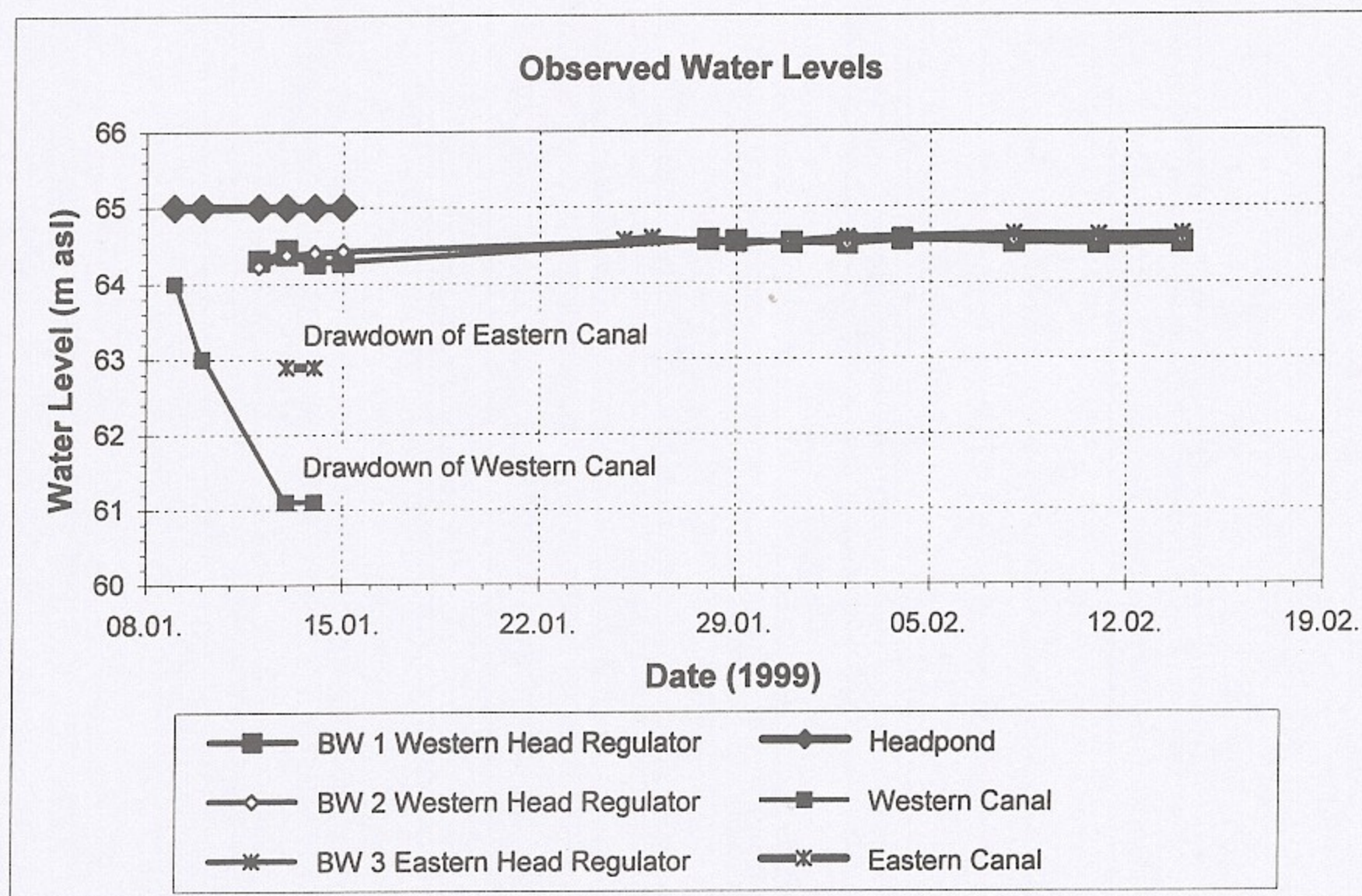
**Figure 8.9: Hydrographs from New Piezometers**

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# APPENDICES

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# Appendix 8.1

# Photo Documentation

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## Appendix 8.1

## Photo Documentation

| Number of photo | Description |
|---|---|

### Western Head Regulator:

| | |
|---|---|
| 1 | View of the upstream part of barrage |
| 2 | View of downstream part of barrage, normal operation (Jan. 99) |
| 3 | View of opening 5 & 6, normal operation (Jan. 99) |
| 4 | View of downstream part of barrage, after complete closure of the gates |
| 5 | Opening 1: View of downstream wall, crack in joint between 3. & 4. block from top |
| 6 | Opening 6: View of downstream wall, crack in joint between 3. & 4. block from top |
| 7 | Right abutment: Shear keys, joints and end of the rail |
| 8 | Downstream end of pier: weathered limestone block (worst case on Western H.R) |
| 9 | Opening 1, Pier 1: Damaged block behind the gate (worst case) |
| 10 | Start of channel downstream of H.R., left side (representative for both sides) |
| 11 | Opening 2: Arch (prefabricated ?), upper and lower gate |
| 12 | Opening 1: View of abutment wall, vertical stripes of washed out material (only on this wall) |
| 13 | Opening 1: View of abutment wall, vertical stripes of washed out material (detail of photo 12) |

### Eastern Head Regulator:

| | |
|---|---|
| 14 | View of the upstream part of barrage |
| 15 | View of downstream part of barrage, after complete closure of the gates |
| 16 | Right abutment, downstream: Damages caused by traffic (worst case) |
| 17 | Opening 1: View of downstream wall, crack in joint between 3. & 4. Block from top |
| 18 | Opening 3: Arch, view from downstream |
| 19 | Left abutment: Horizontal crack on the upstream side, 5 blocks from top |
| 20 | Right abutment: Weathered limestone on the downstream (worst case on Eastern H.R.) |
| 21 | Pier 2: Joint filling missing on the downstream end (only on this pier) |

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## Western Head Regulator



**Photo 1: Upstream View of the Head Regulator**
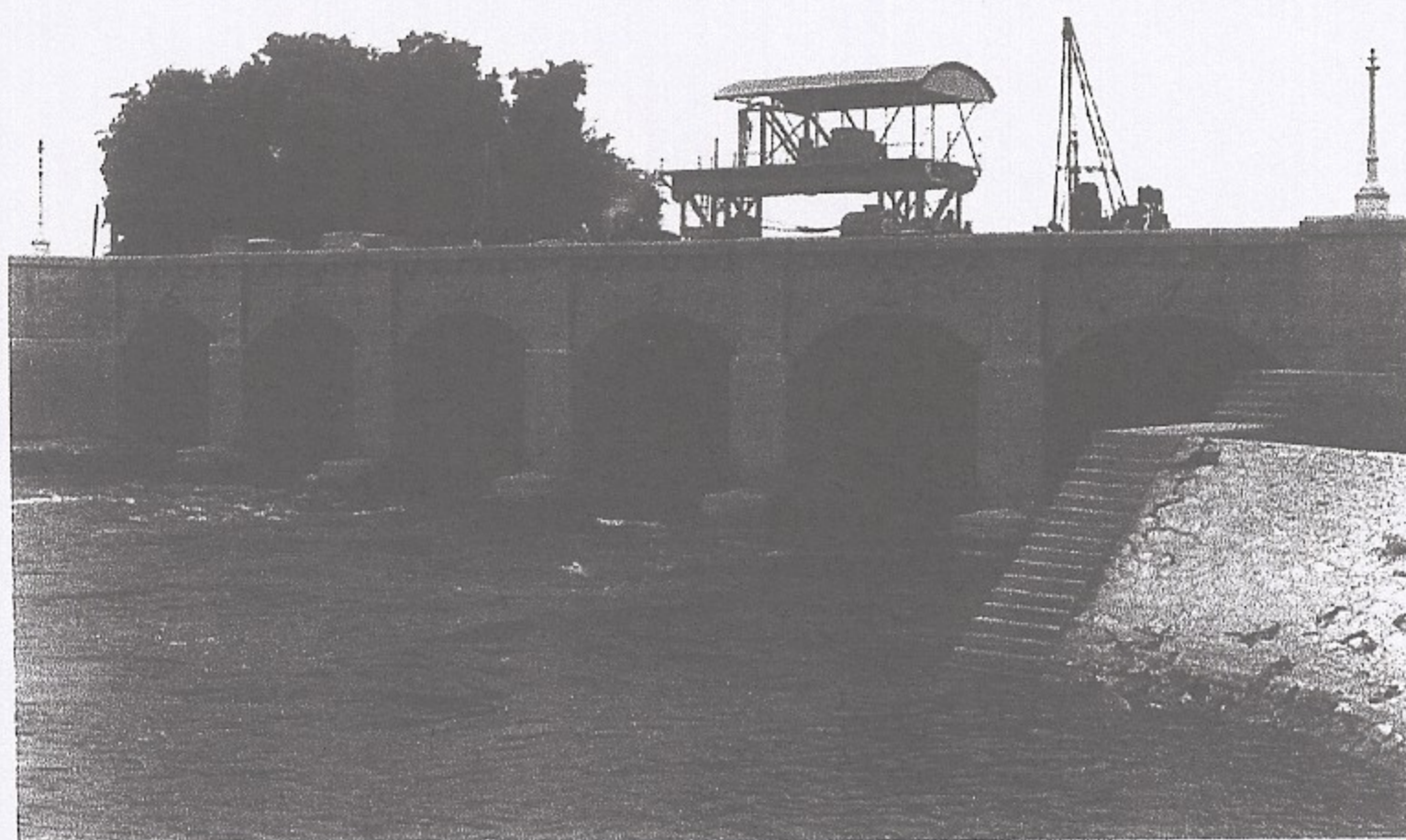


**Photo 2: Downstream View of the Head Regulator, Normal Operation (January 1999)**

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**Photo 3: Downstream View of Openings Nos. 5 & 6, Normal Operation (January 1999)**
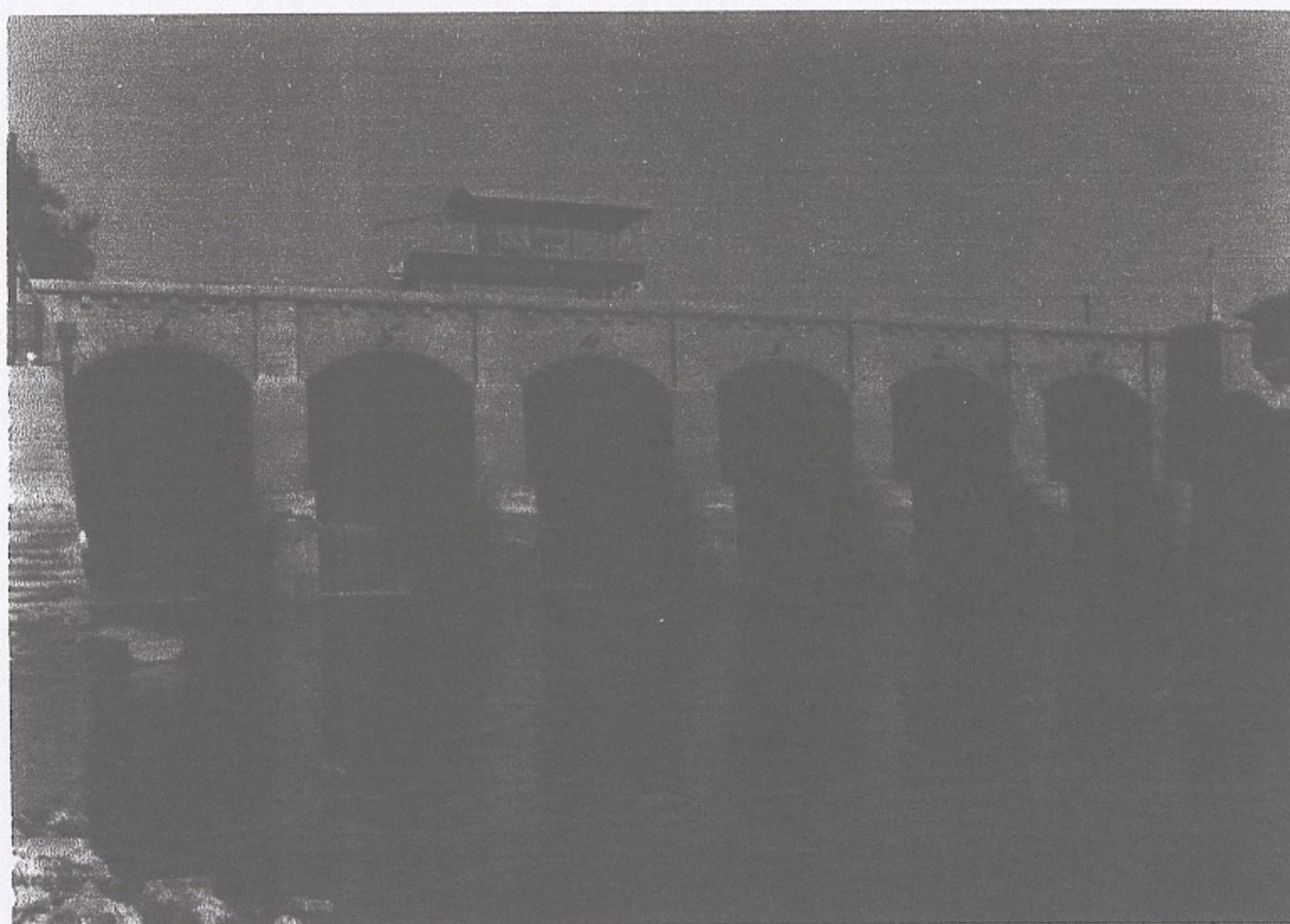


**Photo 4: Downstream View after Closure of the Gates**

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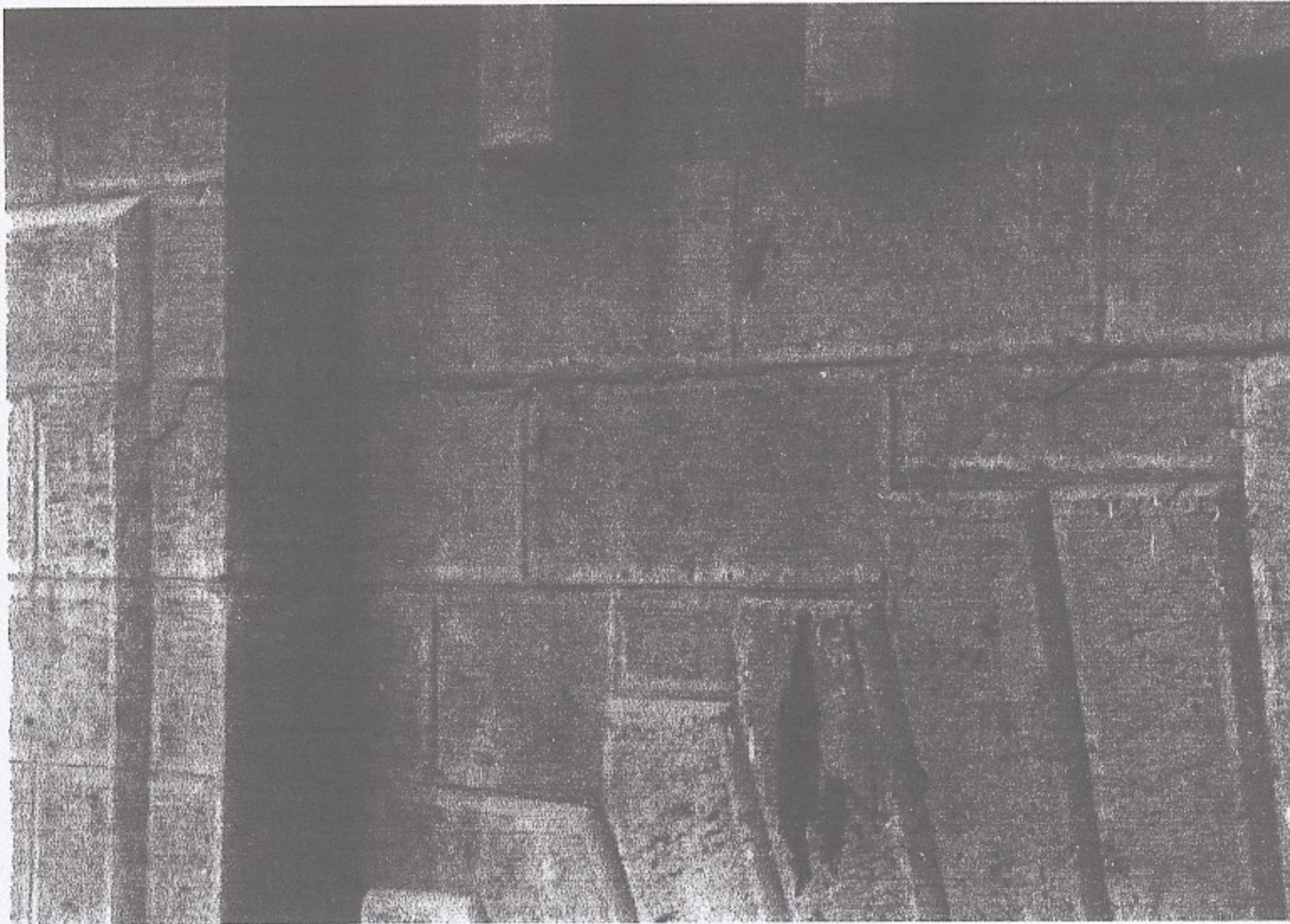
**Photo 5: Downstream View of Wall, Crack in Joint between Blocks Nos. 3 & 4 from Top**
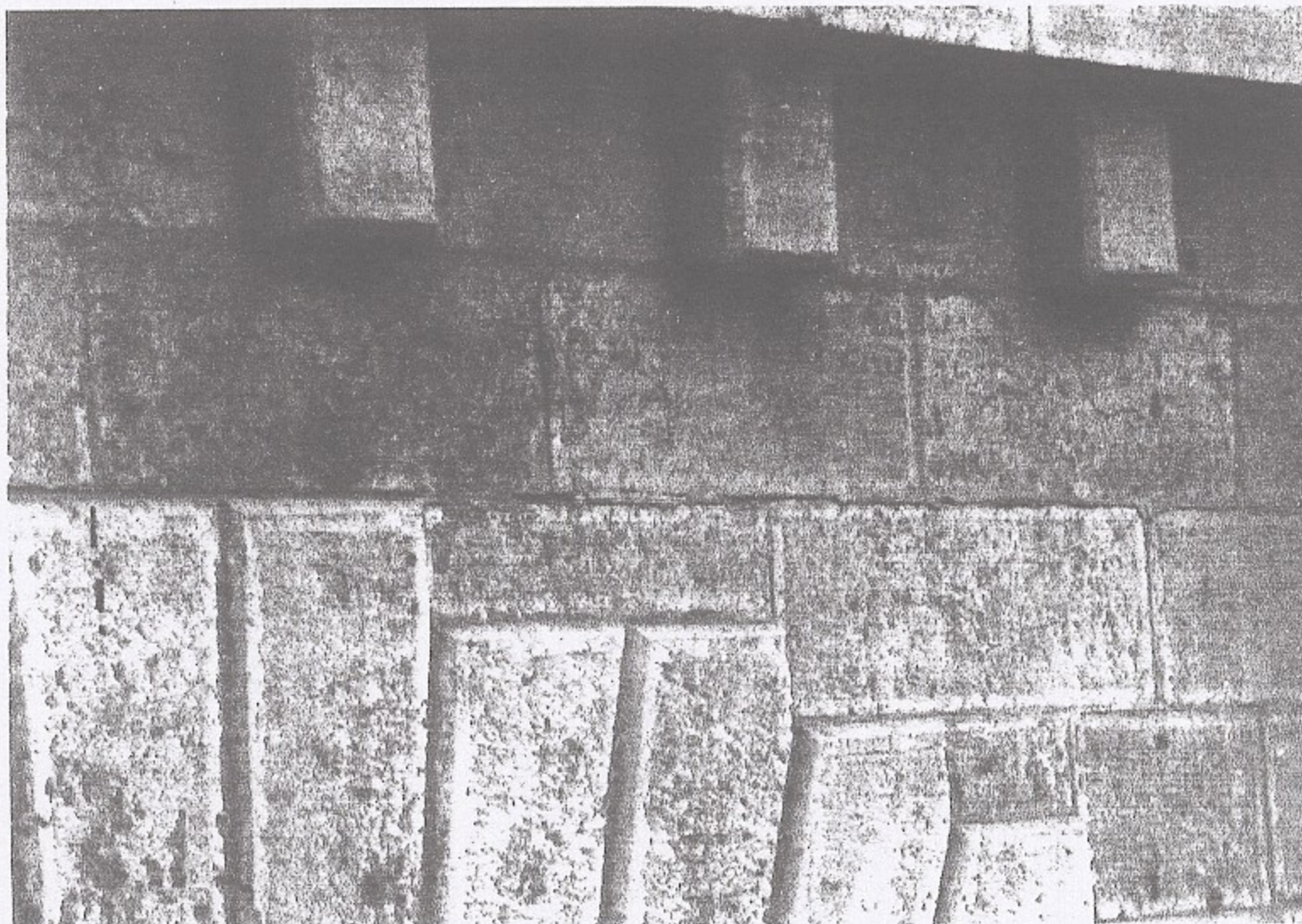


**Photo 6: Opening No. 6 - Downstream View of Wall,
Crack in Joint between Blocks Nos. 3 & 4 from Top**

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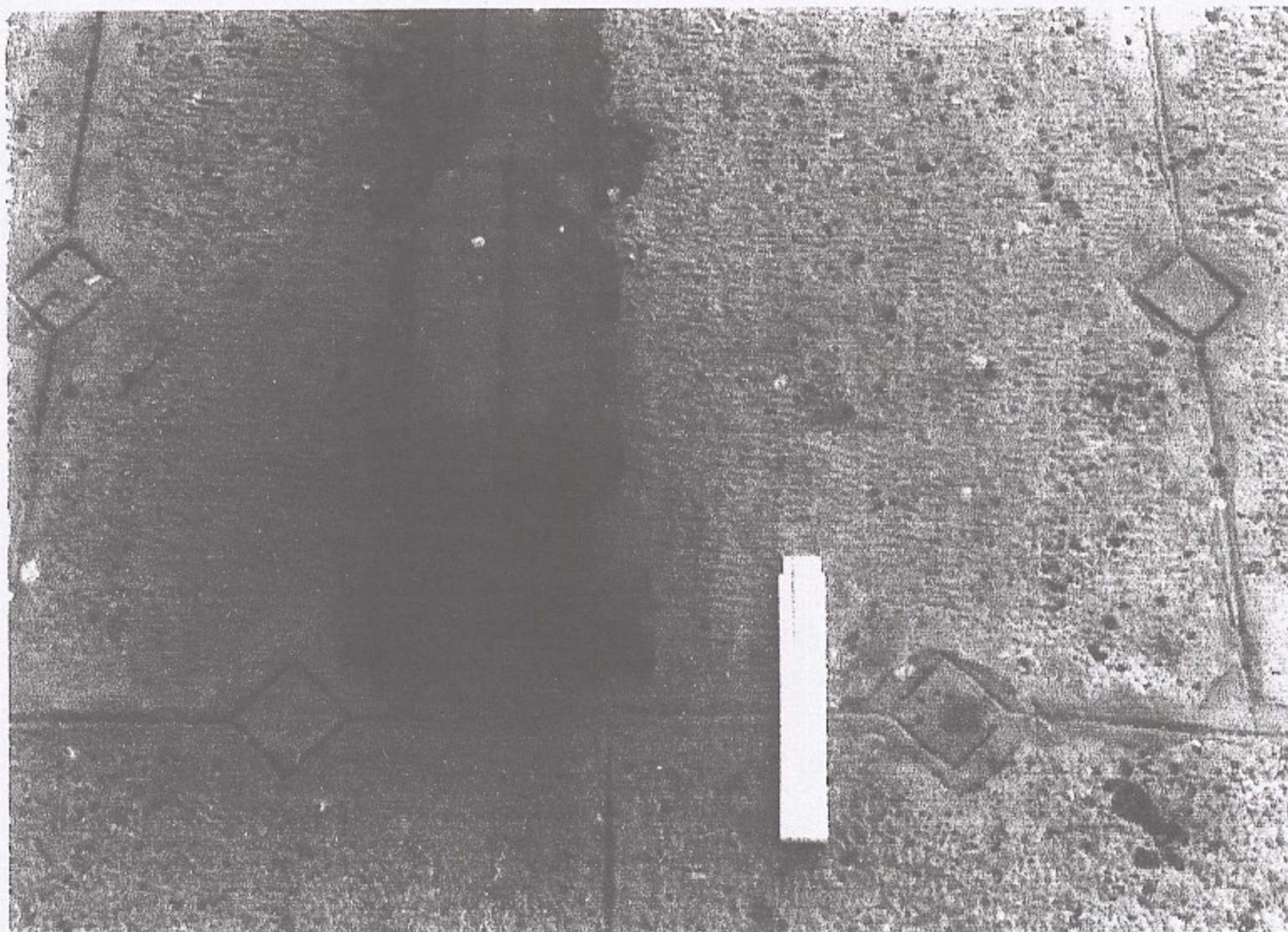
**Photo 7: Right Abutment - Shear Keys, Joints and End of the Rail**



**Photo 8: Downstream End of Pier No. 5 - Weathered Limestone Block
(Worst Case at Western Head Regulator)**

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**Photo 9: Opening No. 1 - Damaged Block behind the Gate (Worst Case)**



**Photo 10: Start of Channel Downstream of Head Regulator, Left Side
(Representative for both Sides)**

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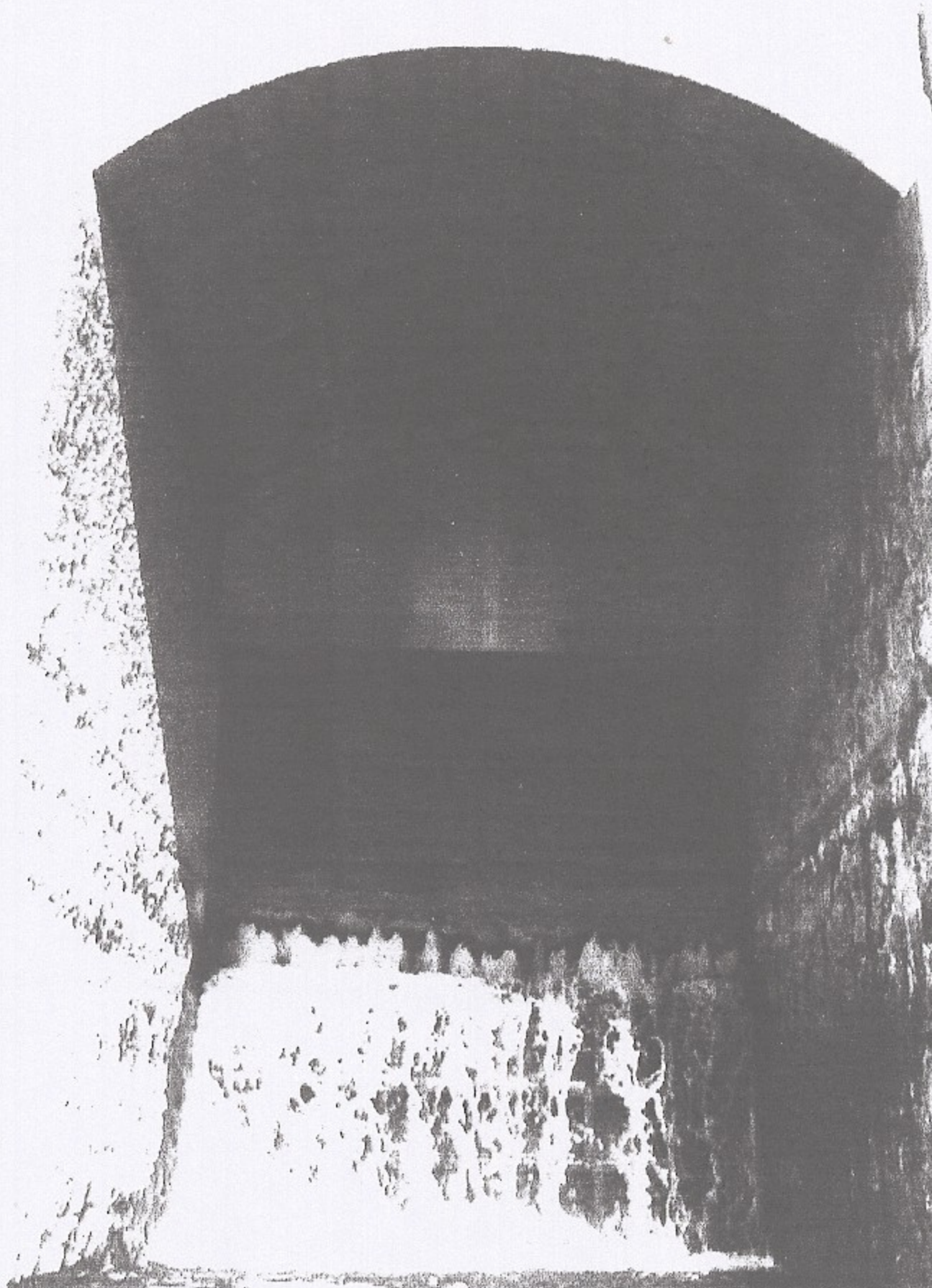


**Photo 11: Opening No. 2 - Arch (prefabricated ?), Upper and Lower Gate**

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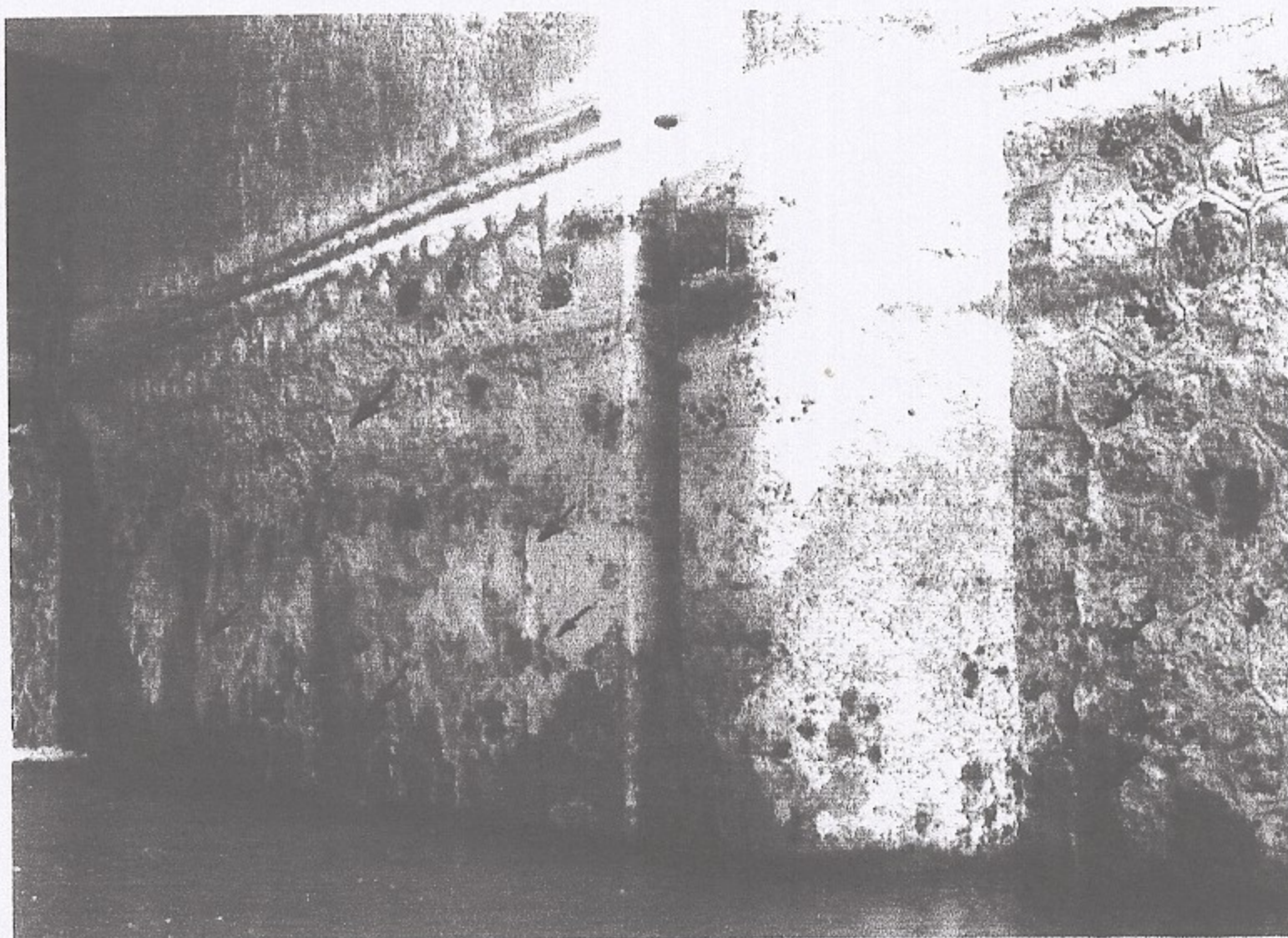
**Photo 12:  Opening No. 1 - View of Abutment Wall,
Vertical Stripes of Washed Out Material (only on this Wall)**



**Photo 13: Opening No. 1 - View of Abutment Wall,
Vertical Stripes of Washed Out Material (Detail of Photo 12)**

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## Eastern Head Regulator
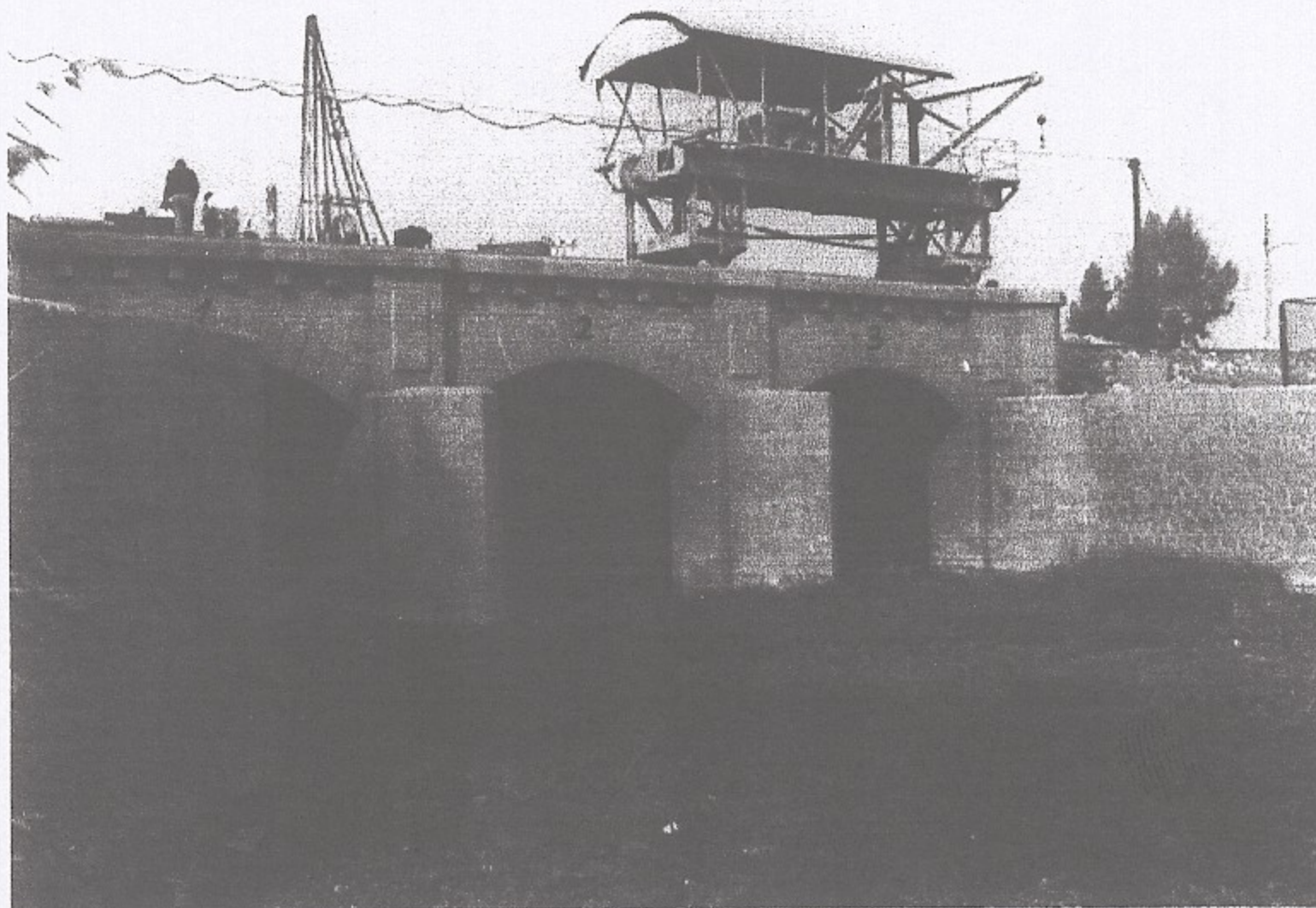


**Photo 14: Upstream View of Head Regulator**



**Photo 15: Downstream View of Head Regulator, after Complete Closure of the Gates**

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**Photo 16: Right Abutment, Downstream - Damages caused by Traffic (Worst Case)**



**Photo 17: Opening No. 1 - View of Downstream Wall,
Crack in Joint between Blocks Nos. 3 & 4 from Top**

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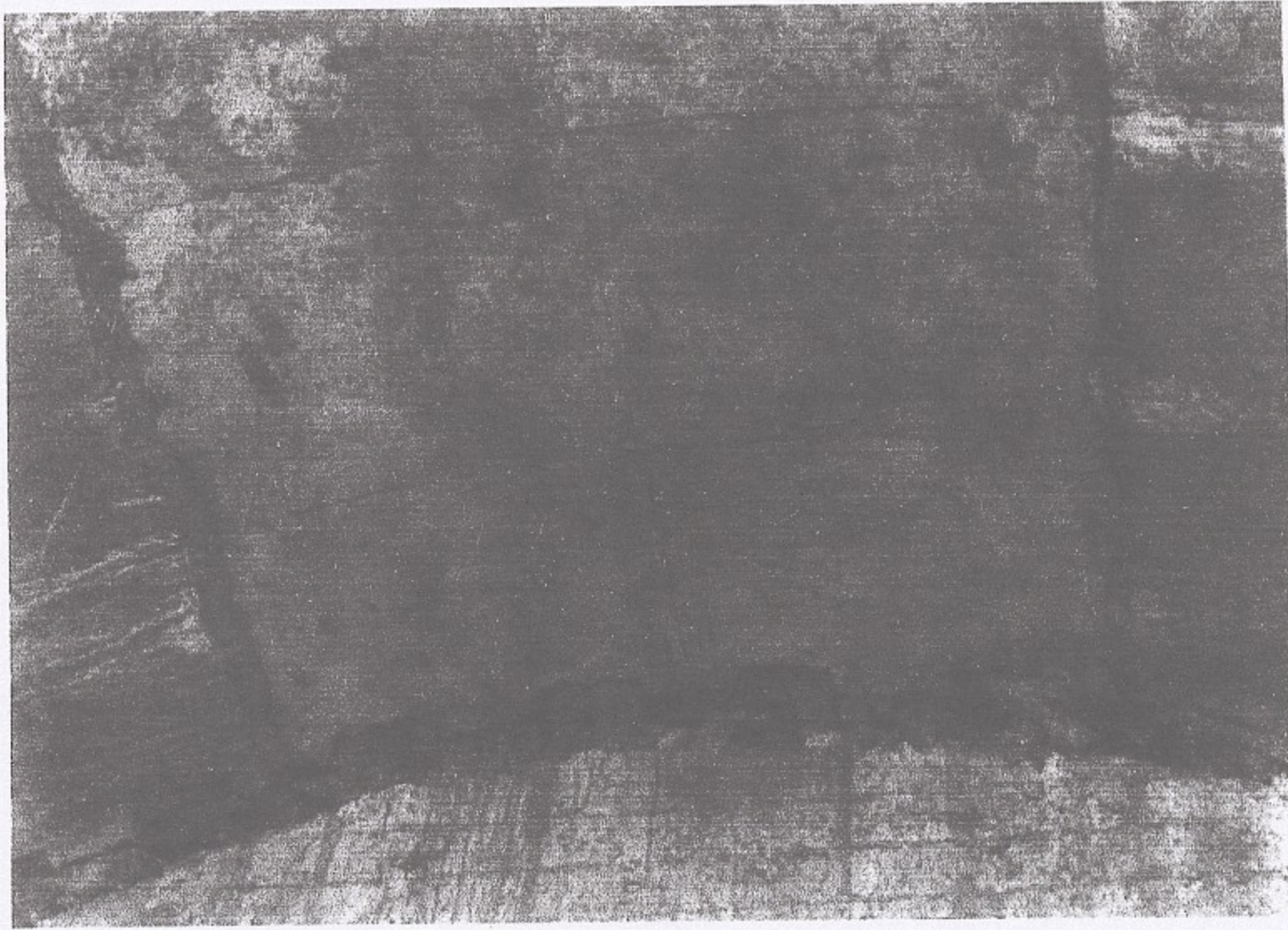
**Photo 18: Opening No. 3 - Arch, Downstream View**
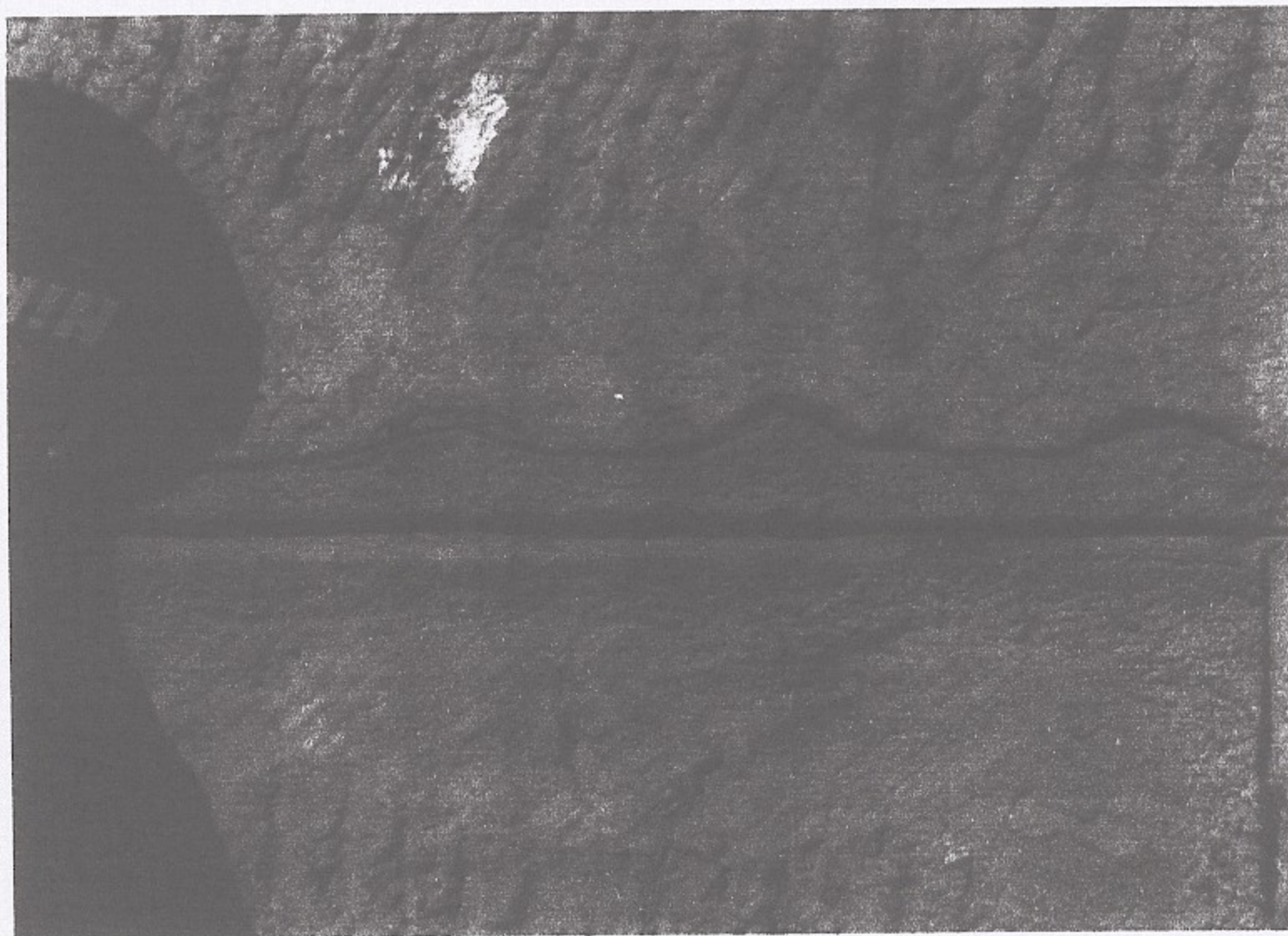


**Photo 19: Left Abutment - Horizontal Crack on the Upstream Side, 5 Blocks from Top**

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**Photo 20: Right Abutment - Weathered Limestone on the Downstream End
(Worst Case at Eastern Head Regulator)**



**Photo 21: Pier No. 2 - Joint Filling missing on the Downstream End (Only at this Pier)**

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# Appendix 8.2

# Stability Analysis

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- Sliding
- Sliding with Earthquake of 0.1 g
- Sand at the End of Slab

## Foundation Slab of the Western Head Regulator

### Summary

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Load Case II:   Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m asl

Load Case III:  Unusual - Existing structure with low tailwater level at 60.50 m asl

Load Case IV:   Unusual - Existing structure, headpond 65.90 m asl, tailwater 60.50 m asl

Load Case V:    Normal - New 75 cm slab, headpond 65.90 m asl, tailwater 64.00 m asl

Load Case VI:   Unusual - New 75 cm slab, headpond 65.90 m asl, tailwater 60.50 m asl

Load Case VII:  Extreme - New 75 cm slab, headpond 67.05 m asl, tailwater 64.00 m asl

## Foundation Slab of the Eastern Head Regulator

### Summary

Load Case I:    Normal - Existing structure headpond 65.10 m asl, tailwater 64.00 m asl

Load Case II:   Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m asl

Load Case III:  Unusual - Existing structure with low tailwater level at 61.25 m asl

Load Case IV:   Unusual - Existing structure, headpond 65.90 m asl, tailwater 61.25 m asl

Load Case V:    Normal - New 75 cm slab, headpond 65.90 m asl, tailwater 64.00 m asl

Load Case VI:   Unusual - New 75 cm slab, headpond 65.90 m asl, tailwater 61.25 m asl

Load Case VII:  Extreme - New 75 cm slab, headpond 67.05 m asl, tailwater 64.00 m asl

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# Foundation Slab of the Western Head Regulator

# Stability Analysis Western Head Regulator

## Summary

| No. | Load Case Water Level Upstream | Water Level Downstream | | Factor of Safety against Uplift at Downstream End | Sliding | Sliding with 10% Earthquake | Uplift 20 m from Downstream | Uplift at Downstream End | Boiling | Piping (Lane) |
|---|---|---|---|---|---|---|---|---|---|---|
| **Existing structure without rehabilitation** | | | | | | | | | | |
| | | | | | | | | | | |
| **Normal Load Case** | | | | | | | | | | |
| I | 65.10 | 64.00 | | 64.64 | 3.79 | 2.02 | 1.37 | 1.40 | 12.5 | 42.5 |
| II | 65.90 | 64.00 | | 65.10 | 3.06 | 1.76 | 1.27 | 1.31 | 7.3 | 24.6 |
| | | | | | | | | | | |
| **Unusual Load Case** | | | | | | | | | | |
| III | 65.10 | 60.50 | | 62.73 | 2.72 | 1.55 | **1.05** | 1.22 | 3.6 | 10.2 |
| IV | 65.90 | 60.50 | | 63.12 | 2.17 | 1.33 | **0.96** | **1.13** | 3.1 | 8.7 |
| | | | | | | | | | | |
| **Rehabilitated structure with new additional 75 cm slab and increased headpond level** | | | | | | | | | | |
| | | | | | | | | | | |
| **Normal Load Case** | | | | | | | | | | |
| V | 65.90 | 64.00 | | 65.10 | 3.13 | 1.79 | 1.50 | 1.56 | 7.3 | 24.6 |
| | | | | | | | | | | |
| **Unusual Load Case** | | | | | | | | | | |
| VI | 65.90 | 60.50 | | 63.12 | 2.25 | 1.36 | 1.24 | 1.46 | 3.1 | 8.7 |
| | | | | | | | | | | |
| **Extreme Load Case** | | | | | | | | | | |
| VII | 67.05 | 64.00 | | 65.77 | 2.36 | 1.47 | 1.35 | 1.43 | 4.5 | 15.3 |
| | | | | | | | | | | |

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# Load Case I:  Normal - Existing structure
## with tailwater level at 64.00 m asl

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# Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case I:**  Normal - Existing structure with tailwater level at 64.00 m asl

| | |
|---|---|
| Water level US: | 65.10 m asl |
| Water level DS: | 64.00 m asl |
| D (H) = | 1.10 m |
| Length of slab: | 51.70 m |
| Bottom of sheet pile wall at the end of slab: | 53.00 m asl |

vertical length at the end was considered with **6.5** weights ==> length of drainage = **123.20** m

Distance of DS Water Level to Bottom of Sheet Pile Wall   11.00 m

| | | | US end of Pier | | Gate Section | | DS end of Pier | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 123.20 | 115.70 | 110.20 | 110.20 | 104.90 | 104.90 | 94.60 | 94.60 | 93.10 | 93.10 | 91.70 | 91.70 | 73.50 | 73.50 | 71.50 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | - | - | - | - | - | | |
| spec. weight (t/m³) | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 4.60 | 4.60 | 4.60 | 4.60 | 4.60 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | |
| Total Weight (t/m²) | 9.07 | 10.23 | 10.23 | 13.76 | 13.76 | 12.66 | 12.66 | 12.58 | 12.58 | 12.58 | 12.58 | 9.70 | 9.70 | 9.70 | 9.70 | |
| water l. under found. for stability analysis | 65.10 | 65.03 | 64.98 | 64.98 | 64.94 | 64.94 | 64.84 | 64.84 | 64.83 | 64.83 | 64.82 | 64.82 | 64.66 | 64.66 | 64.64 | 64.00 |
| Uplift (t/m²) konst.p. | (5.50) | (6.00) | (6.00) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (6.25) | (6.25) | (6.25) | (6.25) | |
| Uplift (t/m²) linear p. | (1.10) | (1.03) | (0.98) | (0.98) | (0.94) | (0.94) | (0.84) | (0.84) | (0.83) | (0.83) | (0.82) | (0.82) | (0.66) | (0.66) | (0.64) | |
| Resulting Uplift | (6.60) | (7.03) | (6.98) | (8.48) | (8.44) | (8.44) | (8.34) | (8.34) | (8.33) | (8.33) | (8.32) | (7.07) | (6.91) | (6.91) | (6.89) | |
| Total | 2.48 | 3.19 | 3.24 | 5.28 | 5.32 | 4.22 | 4.32 | 4.23 | 4.24 | 4.24 | 4.26 | 2.63 | 2.79 | 2.79 | 2.81 | |
| Safety Factor s = | 1.38 | 1.45 | 1.46 | 1.62 | 1.63 | 1.50 | 1.52 | 1.51 | 1.51 | 1.51 | 1.51 | 1.37 | 1.40 | 1.40 | 1.41 | |

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# Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case I:**    Normal - Existing structure with tailwater level at 64.00 m asl

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 13.76 | 8 | | | 583.4 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 12.575 | 8 | | | 291.7 | |
| **Total load** | 18.5 | | | | | **2,677.3** | |
| Uplift | 5.3 | (8.46) | 8 | | | (358.72) | |
| Uplift | 10.3 | (8.39) | 8 | | | (691.39) | |
| Uplift | 2.9 | (8.33) | 8 | | | (193.30) | |
| **Total (Load - Uplift)** | | | | | | **1,433.9** | |
| US water pressure | ( water table at: | | 65.10 m asl) | | | | 295.84 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -225.00 |
| | | | | | | | 70.84 |

Friction angle (°) =     32        =     0.55851
        tg   =     0.625

forces in flow direction:
    US water pressure:                                                    295.84

forces inverse to flow direction:
    DS water pressure                                                     225.00
    tg of load - uplift          1,433.9      * tg  =      895.97
                                                                        1,120.97

| **Safety Factor =** | 1,120.97 | / | 295.84 | = | 3.79 |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Western Head Regulator

**Load Case I:**  Normal - Existing structure with tailwater level at 64.00 m asl

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 13.76 | 8 | | | 583.4 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 12.575 | 8 | | | 291.7 | |
| Total load | 18.5 | | | | | 2,724.8 | |
| Uplift | 5.3 | (8.46) | 8 | | | (358.72) | |
| Uplift | 10.3 | (8.39) | 8 | | | (691.39) | |
| Uplift | 2.9 | (8.33) | 8 | | | (193.30) | |
| Total (Load - Uplift) | | | | | | 1,481.4 | |
| US water pressure | ( water table at: | 65.10 m asl) | | | | | 295.84 |
| add. water pressure (Westergaard) | | | | | | | 0.39 |
| horizontal force from dead load (0.1g) | | | | | | | 272.48 |
| DS water pressure | ( water table at: | 64.00 m asl) | | | | | -225.00 |
| | | | | | | | 343.71 |

Friction angle (°) = 32    =    0.55851
tg  =    0.625

**forces in flow direction:**
US water pressure + Westergaard + dead load 0.1g:    568.71

**forces inverse to flow direction:**
DS water pressure    225.00
tg of load - uplift    1,481.4    * tg =    925.66
1,150.66

| **Safety Factor =** | 1,150.66 | / | 568.71 | = | **2.02** |
|---|---|---|---|---|---|

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# Stability Analysis, Sand at the End of Slab
# Foundation Slab of the Western Head Regulator

**Load Case I:**     Normal - Existing structure with tailwater level at 64.00 m asl

| | |
|---|---|
| **Concrete Slab** | elevation: **61.00** m asl    **Water level:** |
| | Sand:    **upstream:**   65.10 m asl |
| | spec. weight: |
| **Sheet Pile Wall** | **2.0**    **downstr.:**   64.00 m asl |
| | t/m$^3$ |

vertical length at the end was considered with
     **6.5**    weights

elevation:
**53.00**
V

uplift force acting here =      (11.64) t/m$^2$
^ ^ ^ ^ ^ ^ ^

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 t/m$^2$ |
| weight of add. water: | 3.00 t/m$^2$ |
| subtotal vertical forces | 19.00 t/m$^2$ |
| safety factor s =   vertical forces/uplift | 1.63 |
| Total vertical forces = | **7.36** t/m$^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 t/m$^2$ |
| remaining uplift from upstream: | (0.64) t/m$^2$ |
| subtotal | 7.36 t/m$^2$ |

**safety factor** (weight of sand/remain. uplift) **s =**    **12.53**

# Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 1.10 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = **42.48** | > | 6 to 5 |
|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case II:  Normal - Existing structure,
headpond 65.90 m asl, tailwater 64.00 m asl**

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## Stability Analysis, Uplift
### Foundation Slab of the Western Head Regulator

**Load Case II:** Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m asl

| | |
|---|---|
| Water level US: | 65.90 m asl |
| Water level DS: | 64.00 m asl |
| D (H) = | 1.90 m |
| Length of slab: | 51.70 m |
| Bottom of sheet pile wall at the end of slab: | 53.00 m asl |

vertical length at the end was considered with **6.5** weights ==> length of drainage = **123.20** m
Distance of DS Water Level to Bottom of Sheet Pile Wall **11.00** m

| | | | US end of Pier | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Distance from upstream end (m)** | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 123.20 | 115.70 | 110.20 | 110.20 | 104.90 | 104.90 | 94.60 | 94.60 | 93.10 | 93.10 | 91.70 | 91.70 | 73.50 | 73.50 | 71.50 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | - | - | - | - | - | | | |
| spec. weight (t/m³) | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | | |
| water load (t/m²) | 5.40 | 5.40 | 5.40 | 5.40 | 5.40 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | |
| **Total Weight (t/m²)** | 9.88 | 11.03 | 11.03 | 14.56 | 14.56 | 12.66 | 12.66 | 12.58 | 12.58 | 12.58 | 12.58 | 9.70 | 9.70 | 9.70 | 9.70 | |
| water l. under found. for stability analysis | 65.90 | 65.78 | 65.70 | 65.70 | 65.62 | 65.62 | 65.46 | 65.46 | 65.44 | 65.44 | 65.41 | 65.41 | 65.13 | 65.13 | 65.10 | 64.00 |
| Uplift (t/m²) konst.p. | (5.50) | (6.00) | (6.00) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (6.25) | (6.25) | (6.25) | (6.25) | |
| Uplift (t/m²) linear p. | (1.90) | (1.78) | (1.70) | (1.70) | (1.62) | (1.62) | (1.46) | (1.46) | (1.44) | (1.44) | (1.41) | (1.41) | (1.13) | (1.13) | (1.10) | |
| **Resulting Uplift** | (7.40) | (7.78) | (7.70) | (9.20) | (9.12) | (9.12) | (8.96) | (8.96) | (8.94) | (8.94) | (8.91) | (7.66) | (7.38) | (7.38) | (7.35) | |
| **Total** | 2.48 | 3.24 | 3.33 | 5.36 | 5.44 | 3.54 | 3.70 | 3.62 | 3.64 | 3.64 | 3.66 | 2.04 | 2.32 | 2.32 | 2.35 | |
| **Safety Factor s =** | 1.33 | 1.42 | 1.43 | 1.58 | 1.60 | 1.39 | 1.41 | 1.40 | 1.41 | 1.41 | 1.41 | 1.27 | 1.31 | 1.31 | 1.32 | |



Chart legend: top level of old slab — bottom elevation of top material — bottom elevation of the concrete — top level of new slab — Water Level — resulting Uplift — piezometric head

Upper chart: Elevation (m asl) vs Distance from Upstream End (m)

Lower chart: Factor of Safety vs Distance from Upstream End (m)

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# Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case II:**     Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m as

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 12.575 | 8 | | | 291.7 | |
| **Total load** | 18.5 | | | | | 2,711.2 | |
| Uplift | 5.3 | (9.16) | 8 | | | (388.33) | |
| Uplift | 10.3 | (9.04) | 8 | | | (744.76) | |
| Uplift | 2.9 | (8.94) | 8 | | | (207.33) | |
| **Total (Load - Uplift)** | | | | | | 1,370.8 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 353.44 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -225.00 |
| | | | | | | | 128.44 |

Friction angle (°) =     32     =     0.55851
         tg  =     0.625

**forces in flow direction:**
         US water pressure:                          353.44

**forces inverse to flow direction:**
         DS water pressure                          225.00
         tg of load - uplift          1,370.8     * tg  =     856.54
                                                        1,081.54

| **Safety Factor =** | 1,081.54 | / | 353.44 | = | **3.06** |
|---|---|---|---|---|---|

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## Stability Analysis, Sliding with Earthquake of 0.1 g
### Foundation Slab of the Western Head Regulator

**Load Case II:**   Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 12.575 | 8 | | | 291.7 | |
| Total load | 18.5 | | | | | 2,758.7 | |
| Uplift | 5.3 | (9.16) | 8 | | | (388.33) | |
| Uplift | 10.3 | (9.04) | 8 | | | (744.76) | |
| Uplift | 2.9 | (8.94) | 8 | | | (207.33) | |
| Total (Load - Uplift) | | | | | | 1,418.3 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 353.44 |
| add. water pressure (Westergaard) | | | | | | | 1.18 |
| horizontal force from dead load (0.1g) | | | | | | | 275.87 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -225.00 |
| | | | | | | | 405.49 |

Friction angle (°) =         32            =         0.55851
         tg   =      0.625

**forces in flow direction:**
         US water pressure + Westergaard + dead load 0.1g:         630.49

**forces inverse to flow direction:**
         DS water pressure                                                      225.00
         tg of load - uplift                          1,418.3       * tg  =      886.24
                                                                                   1,111.24

| **Safety Factor =** | 1,111.24 | / | 630.49 | = | **1.76** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case II:** Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m asl

|  |  |  |
|---|---|---|
| 65.9 | | |
| 64 | **61.00** m asl | **Water level:** |

**Concrete Slab**

Sand:
spec. weight:
**2.0**
t/m³

**upstream:** 65.90 m asl

**downstr.:** 64.00 m asl

**Sheet Pile Wall**

vertical length at the end was considered with
**6.5** weights

elevation:
**53.00**
V

uplift force acting here = (12.10) t/m²

∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 t/m² |
| weight of add. water: | 3.00 t/m² |
| subtotal vertical forces | 19.00 t/m² |
| safety factor s = vertical forces/uplift | 1.57 |
| Total vertical forces = | **6.90** t/m² |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 t/m² |
| remaining uplift from upstream: | (1.10) t/m² |
| subtotal | 6.90 t/m² |
| **safety factor** (weight of sand/remain. uplift) s = | **7.26** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 1.90 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | **24.60** | > | **6 to 5** |
|---|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :
- Very fine sand or silt to fine sand — 8.5 to 7
- **Medium to coarse sand** — **6 to 5**
- Fine to coarse gravel — 4 to 3
- Boulders with some cobbles and gravel — 2.5
- Soft to medium clay — 3 to 2
- Hard to very hard clay — 1.8 to 1.6

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**Load Case III: Unusual - Existing structure**

**with low tailwater level at 60.50 m asl**

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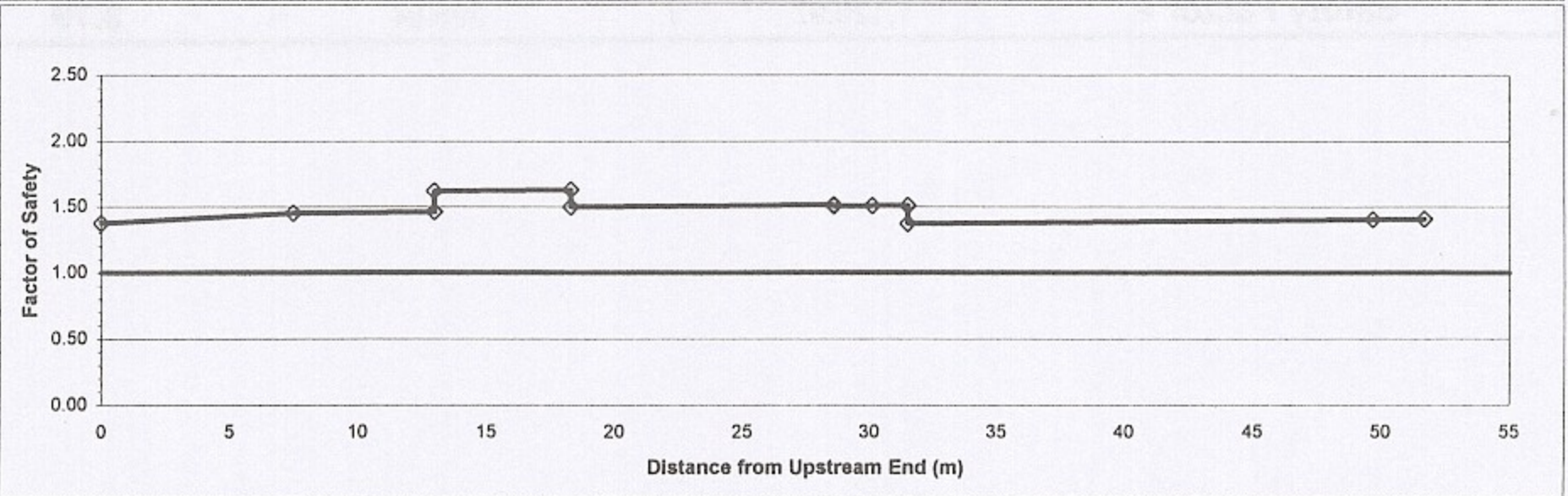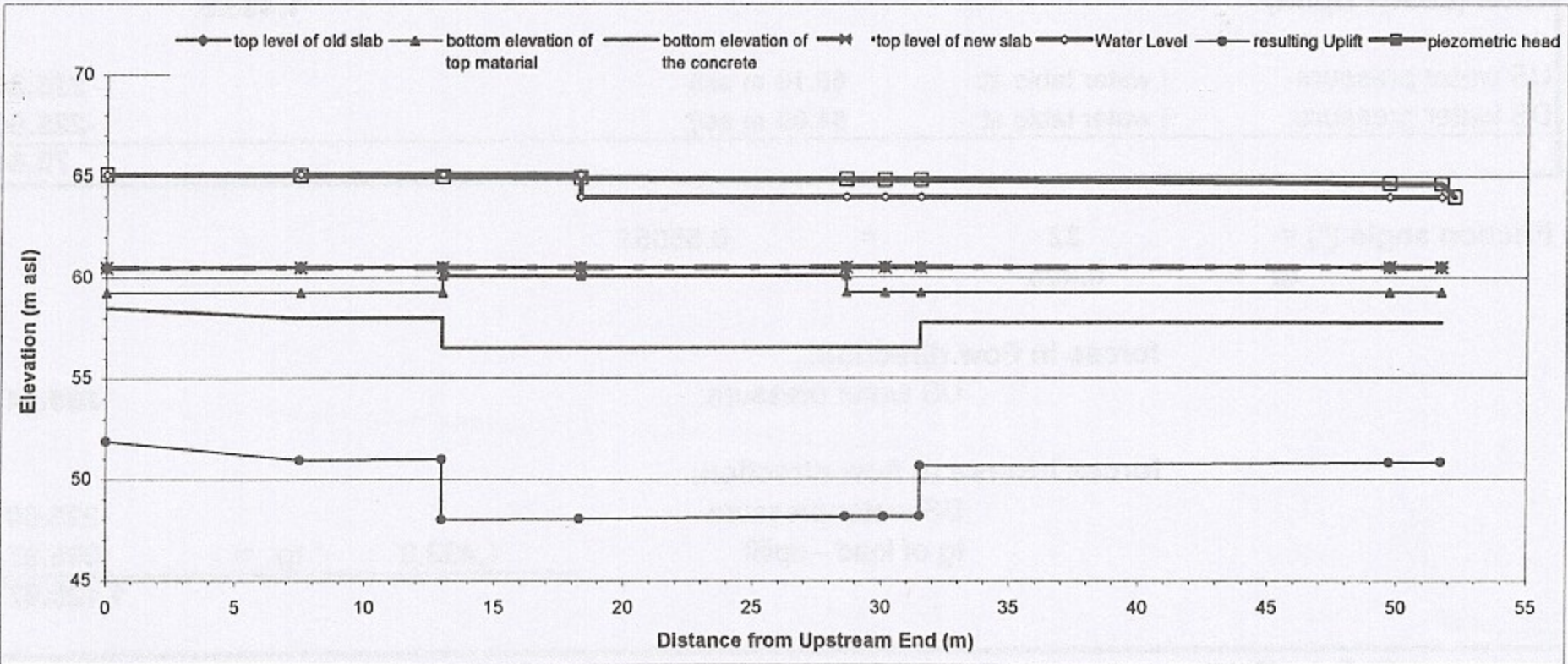Tender Design Report
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# Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case III:** Unusual - Existing structure with low tailwater level at 60.50 m asl

| | |
|---|---|
| Water level US: | 65.10 m asl |
| Water level DS: | 60.50 m asl |
| D (H) = | 4.60 m |
| Length of slab: | 51.70 m |
| Bottom of sheet pile wall at the end of slab: | 53.00 m asl |

vertical length at the end was considered with 6.5 weights ==> length of drainage = 100.45 m

Distance of DS Water Level to Bottom of Sheet Pile Wall 7.50 m

| | | US end of Pier | | | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Distance from upstream end (m)** | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 100.45 | 92.95 | 87.45 | 87.45 | 82.15 | 82.15 | 71.85 | 71.85 | 70.35 | 70.35 | 68.95 | 68.95 | 50.75 | 50.75 | 48.75 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | - | - | - | - | - | - | - | - | | | |
| spec. weight (t/m³) | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | | |
| water load (t/m²) | 4.60 | 4.60 | 4.60 | 4.60 | 4.60 | - | - | - | - | - | - | - | - | - | - | |
| Total Weight (t/m²) | 9.07 | 10.23 | 10.23 | 13.76 | 13.76 | 9.16 | 9.16 | 9.08 | 9.08 | 9.08 | 9.08 | 6.20 | 6.20 | 6.20 | 6.20 | |
| water l. under found. for stability analysis | 65.10 | 64.76 | 64.50 | 64.50 | 64.26 | 64.26 | 63.79 | 63.79 | 63.72 | 63.72 | 63.66 | 63.66 | 62.82 | 62.82 | 62.73 | 60.50 |
| Uplift (t/m²) konst.p. | (2.00) | (2.50) | (2.50) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (2.75) | (2.75) | (2.75) | (2.75) | |
| Uplift (t/m²) linear p. | (4.60) | (4.26) | (4.00) | (4.00) | (3.76) | (3.76) | (3.29) | (3.29) | (3.22) | (3.22) | (3.16) | (3.16) | (2.32) | (2.32) | (2.23) | |
| Resulting Uplift | (6.60) | (6.76) | (6.50) | (8.00) | (7.76) | (7.76) | (7.29) | (7.29) | (7.22) | (7.22) | (7.16) | (5.91) | (5.07) | (5.07) | (4.98) | |
| Total | 2.48 | 3.47 | 3.72 | 5.76 | 6.00 | 1.40 | 1.87 | 1.78 | 1.85 | 1.85 | 1.92 | 0.29 | 1.13 | 1.13 | 1.22 | |
| Safety Factor s = | 1.38 | 1.51 | 1.57 | 1.72 | 1.77 | 1.18 | 1.26 | 1.24 | 1.26 | 1.26 | 1.27 | 1.05 | 1.22 | 1.22 | 1.24 | |

Legend: top level of old slab · bottom elevation of top material · bottom elevation of the concrete · top level of new slab · Water Level · resulting Uplift · piezometric head

Chart 1 — Y-axis: Elevation (m asl) (70, 65, 60, 55, 50, 45); X-axis: Distance from Upstream End (m) (0 to 55)

Chart 2 — Y-axis: Factor of Safety (0.00 to 2.50); X-axis: Distance from Upstream End (m) (0 to 55)

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# Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case III:**      Unusual - Existing structure with low tailwater level at 60.50 m asl

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 13.76 | 8 | | | 583.4 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 9.075 | 8 | | | 210.5 | |
| Total load | 18.5 | | | | | 2,307.7 | |
| Uplift | 5.3 | (7.88) | 8 | | | (334.25) | |
| Uplift | 10.3 | (7.53) | 8 | | | (620.15) | |
| Uplift | 2.9 | (7.22) | 8 | | | (167.59) | |
| Total (Load - Uplift) | | | | | | 1,185.7 | |
| US water pressure | ( water table at: | 65.10 m asl) | | | | | 295.84 |
| DS water pressure | ( water table at: | 60.50 m asl) | | | | | -64.00 |
| | | | | | | | 231.84 |

Friction angle (°) =    32   =   0.55851
     tg  =   0.625

forces in flow direction:
     US water pressure:        295.84

forces inverse to flow direction:
     DS water pressure        64.00
     tg of load - uplift      1,185.7    * tg  =    740.88
                         804.88

| Safety Factor = | 804.88 | / | 295.84 | = | **2.72** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Western Head Regulator

**Load Case III:**     Unusual - Existing structure with low tailwater level at 60.50 m asl

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | height x spec.w. | | | | | | |
| found. slab | 5.3 | 13.76 | 8 | | | 583.4 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 9.075 | 8 | | | 210.5 | |
| **Total load** | 18.5 | | | | | **2,355.2** | |
| Uplift | 5.3 | (7.88) | 8 | | | (334.25) | |
| Uplift | 10.3 | (7.53) | 8 | | | (620.15) | |
| Uplift | 2.9 | (7.22) | 8 | | | (167.59) | |
| **Total (Load - Uplift)** | | | | | | **1,233.2** | |
| US water pressure | ( water table at: | | 65.10 m asl) | | | | 295.84 |
| add. water pressure (Westergaard) | | | | | | | 6.91 |
| horizontal force from dead load (0.1g) | | | | | | | 235.52 |
| DS water pressure | ( water table at: | | 60.50 m asl) | | | | -64.00 |
| | | | | | | | 474.26 |

Friction angle (°) =          32          =          0.55851
          tg  =     0.625

forces in flow direction:

          US water pressure + Westergaard + dead load 0.1g:          538.26

forces inverse to flow direction:

          DS water pressure                                             64.00
          tg of load - uplift          1,233.2     * tg  =          770.57
                                                                    834.57

| **Safety Factor =** | 834.57 | / | 538.26 | = | **1.55** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case III:**   Unusual - Existing structure with low tailwater level at 60.50 m asl

| | |
|---|---|
| **Concrete Slab** | elevation: **61.00** m asl |
| | Sand:  spec. weight: **2.0** t/m³ |
| **Sheet Pile Wall** | |
| elevation: **53.00** | |

**Water level:**

upstream:   **65.10**  m asl

downstr.:   **60.50**  m asl

vertical length at the end was considered with **6.5** weights

uplift force acting here = **(9.73)** t/m²

∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 t/m² |
| weight of add. water: | -0.50 t/m² |
| subtotal vertical forces | 15.50 t/m² |
| safety factor s = vertical forces/uplift | 1.59 |
| Total vertical forces = | **5.77** t/m² |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 t/m² |
| remaining uplift from upstream: | (2.23) t/m² |
| subtotal | 5.77 t/m² |
| **safety factor** (weight of sand/remain. uplift) **s =** | **3.58** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 4.60 | |

**Weighted Creep Ratio WCR =**   $( Dv + Dh / 3 ) / H =$   **10.16**   **>**   **6 to 5**

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case IV:  Unusual - Existing structure,
headpond 65.90 m asl, tailwater 60.50 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case IV:** Unusual - Existing Structure, Headpond 65.90 m asl, tailwater 60.50 m asl

| | |
|---|---|
| Water level US: | 65.90 m asl |
| Water level DS: | 60.50 m asl |
| D (H) = | 5.40 m |
| Length of slab: | 51.70 m |
| Bottom of sheet pile wall at the end of slab: | 53.00 m asl |

vertical length at the end was considered with **6.5** weights ==> length of drainage = **100.45** m
Distance of DS Water Level to Bottom of Sheet Pile Wall **7.50** m

| | US end of Pier | | | Gate Section | | | | DS end of Pier | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 100.45 | 92.95 | 87.45 | 87.45 | 82.15 | 82.15 | 71.85 | 71.85 | 70.35 | 70.35 | 68.95 | 68.95 | 50.75 | 50.75 | 48.75 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | - | - | - | - | - | | | |
| spec. weight (t/m³) | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | | |
| water load (t/m²) | 5.40 | 5.40 | 5.40 | 5.40 | 5.40 | - | - | - | - | - | - | - | - | - | - | |
| Total Weight (t/m²) | 9.88 | 11.03 | 11.03 | 14.56 | 14.56 | 9.16 | 9.16 | 9.08 | 9.08 | 9.08 | 9.08 | 6.20 | 6.20 | 6.20 | 6.20 | |
| water l. under found. for stability analysis | 65.90 | 65.50 | 65.20 | 65.20 | 64.92 | 64.92 | 64.36 | 64.36 | 64.28 | 64.28 | 64.21 | 64.21 | 63.23 | 63.23 | 63.12 | 60.50 |
| Uplift (t/m²) konst.p. | (2.00) | (2.50) | (2.50) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (2.75) | (2.75) | (2.75) | (2.75) | |
| Uplift (t/m²) linear p. | (5.40) | (5.00) | (4.70) | (4.70) | (4.42) | (4.42) | (3.86) | (3.86) | (3.78) | (3.78) | (3.71) | (3.71) | (2.73) | (2.73) | (2.62) | |
| Resulting Uplift | (7.40) | (7.50) | (7.20) | (8.70) | (8.42) | (8.42) | (7.86) | (7.86) | (7.78) | (7.78) | (7.71) | (6.46) | (5.48) | (5.48) | (5.37) | |
| Total | 2.48 | 3.53 | 3.82 | 5.86 | 6.14 | 0.74 | 1.30 | 1.21 | 1.29 | 1.29 | 1.37 | (0.26) | 0.72 | 0.72 | 0.83 | |
| Safety Factor s = | 1.33 | 1.47 | 1.53 | 1.67 | 1.73 | 1.09 | 1.17 | 1.15 | 1.17 | 1.17 | 1.18 | 0.96 | 1.13 | 1.13 | 1.15 | |

Chart 1 — legend: top level of old slab · bottom elevation of top material · bottom elevation of the concrete · top level of new slab · Water Level · resulting Uplift · piezometric head
Y-axis: Elevation (m asl), 45 – 70
X-axis: Distance from Upstream End (m), 0 – 55

Chart 2 — Y-axis: Factor of Safety, 0.00 – 2.50
X-axis: Distance from Upstream End (m), 0 – 55

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# Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case IV:** Unusual - Existing Structure, Headpond 65.90 m asl, tailwater 60.50 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| **Total Pier** | | | | 345.0 | | 758.9 | |
| | | **height x spec.w.** | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 9.075 | 8 | | | 210.5 | |
| **Total load** | 18.5 | | | | | **2,341.6** | |
| Uplift | 5.3 | (8.56) | 8 | | | (362.89) | |
| Uplift | 10.3 | (8.14) | 8 | | | (670.68) | |
| Uplift | 2.9 | (7.78) | 8 | | | (180.60) | |
| **Total (Load - Uplift)** | | | | | | 1,127.4 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 353.44 |
| DS water pressure | ( water table at: | | 60.50 m asl) | | | | -64.00 |
| | | | | | | | 289.44 |

Friction angle (°) =  32   =  0.55851

tg  =  0.625

forces in flow direction:
US water pressure:  353.44

forces inverse to flow direction:
DS water pressure  64.00
tg of load - uplift   1,127.4   * tg =   704.48
768.48

**Safety Factor =**   768.48   /   353.44   =   **2.17**

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Western Head Regulator

**Load Case IV:**  Unusual - Existing Structure, Headpond 65.90 m asl, tailwater 60.50 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume $m^3$ | spec. weight $t/m^3$ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 9.075 | 8 | | | 210.5 | |
| Total load | 18.5 | | | | | 2,389.1 | |
| Uplift | 5.3 | (8.56) | 8 | | | (362.89) | |
| Uplift | 10.3 | (8.14) | 8 | | | (670.68) | |
| Uplift | 2.9 | (7.78) | 8 | | | (180.60) | |
| Total (Load - Uplift) | | | | | | 1,174.9 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 353.44 |
| add. water pressure (Westergaard) | | | | | | | 9.52 |
| horizontal force from dead load (0.1g) | | | | | | | 238.91 |
| DS water pressure | ( water table at: | | 60.50 m asl) | | | | -64.00 |
| | | | | | | | 537.87 |

Friction angle (°) =     32          =          0.55851

tg  =     0.625

forces in flow direction:

US water pressure + Westergaard + dead load 0.1g:     **601.87**

forces inverse to flow direction:

DS water pressure     **64.00**

tg of load - uplift     1,174.9     * tg  =     **734.17**

**798.17**

| **Safety Factor =** | 798.17 | / | 601.87 | = | **1.33** |
|---|---|---|---|---|---|

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# Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case IV:**  Unusual - Existing Structure, Headpond 65.90 m asl, tailwater 60.50 m asl

**Concrete Slab**

elevation:  **61.00**  m asl

**Water level:**

Sand:
spec. weight:
**2.0**
t/m$^3$

**upstream:**  **65.90**  m asl

**downstr.:**  **60.50**  m asl

**Sheet Pile Wall**

vertical length at the end was considered with
**6.5**  weights

elevation:
**53.00**
V

uplift force acting here =  (10.12) t/m$^2$
^ ^ ^ ^ ^ ^ ^

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 t/m$^2$ |
| weight of add. water: | -0.50 t/m$^2$ |
| subtotal vertical forces | 15.50 t/m$^2$ |
| safety factor s =  vertical forces/uplift | 1.53 |
| Total vertical forces = | **5.38** t/m$^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 t/m$^2$ |
| remaining uplift from upstream: | (2.62) t/m$^2$ |
| subtotal | 5.38 t/m$^2$ |
| **safety factor** (weight of sand/remain. uplift) s = | **3.05** |

# Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 5.40 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | **8.65** | > | **6 to 5** |
|---|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| **- Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case V: Normal - New 75 cm slab,
headpond 65.90 m asl, tailwater 64.00 m asl**

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## Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case V:** Normal - New 75 cm Slab, Headpond 65.90 m asl, tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 1.90 | m |
| Length of slab: | 51.70 | m |
| Bottom of sheet pile wall at the end of slab: | 53.00 | m asl |

vertical length at the end was considered with 6.5 weights ==> length of drainage = 123.20 m
Distance of DS Water Level to Bottom of Sheet Pile Wall 11.00 m

| | | | US end of Pier | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 123.20 | 115.70 | 110.20 | 110.20 | 104.90 | 104.90 | 94.60 | 94.60 | 93.10 | 93.10 | 91.70 | 91.70 | 73.50 | 73.50 | 71.50 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | - | | |
| spec. weight (t/m³) | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 5.40 | 5.40 | 5.40 | 5.40 | 5.40 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | |
| Total Weight (t/m²) | 9.88 | 11.03 | 11.03 | 14.56 | 14.56 | 12.66 | 12.66 | 12.58 | 12.58 | 14.38 | 14.38 | 11.50 | 11.50 | 9.70 | 9.70 | |
| water l. under found. for stability analysis | 65.90 | 65.78 | 65.70 | 65.70 | 65.62 | 65.62 | 65.46 | 65.46 | 65.44 | 65.44 | 65.41 | 65.41 | 65.13 | 65.13 | 65.10 | 64.00 |
| Uplift (t/m²) konst.p. | (5.50) | (6.00) | (6.00) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (6.25) | (6.25) | (6.25) | (6.25) | |
| Uplift (t/m²) linear p. | (1.90) | (1.78) | (1.70) | (1.70) | (1.62) | (1.62) | (1.46) | (1.46) | (1.44) | (1.44) | (1.41) | (1.41) | (1.13) | (1.13) | (1.10) | |
| Resulting Uplift | (7.40) | (7.78) | (7.70) | (9.20) | (9.12) | (9.12) | (8.96) | (8.96) | (8.94) | (8.94) | (8.91) | (7.66) | (7.38) | (7.38) | (7.35) | |
| Total | 2.48 | 3.24 | 3.33 | 5.36 | 5.44 | 3.54 | 3.70 | 3.62 | 3.64 | 5.44 | 5.46 | 3.84 | 4.12 | 2.32 | 2.35 | |
| Safety Factor s = | 1.33 | 1.42 | 1.43 | 1.58 | 1.60 | 1.39 | 1.41 | 1.40 | 1.41 | 1.61 | 1.61 | 1.50 | 1.56 | 1.31 | 1.32 | |

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## Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case V:**     Normal - New 75 cm Slab, Headpond 65.90 m asl, tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 14.375 | 8 | | | 333.5 | |
| **Total load** | 18.5 | | | | | 2,752.9 | |
| Uplift | 5.3 | (9.16) | 8 | | | (388.33) | |
| Uplift | 10.3 | (9.04) | 8 | | | (744.76) | |
| Uplift | 2.9 | (8.94) | 8 | | | (207.33) | |
| **Total (Load - Uplift)** | | | | | | 1,412.5 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 353.44 |
| DS water pressure | ( water table at: | 64.00 m asl) | | | | | -225.00 |
| | | | | | | | 128.44 |

Friction angle (°) =     32     =     0.55851
          tg =     0.625

forces in flow direction:
          US water pressure:                                              353.44

forces inverse to flow direction:
          DS water pressure                                              225.00
          tg of load - uplift          1,412.5     * tg =          882.64
                                                                        1,107.64

| **Safety Factor =** | 1,107.64 | / | 353.44 | = | **3.13** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Western Head Regulator

**Load Case V:**  Normal - New 75 cm Slab, Headpond 65.90 m asl, tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | height x spec.w. | | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 14.375 | 8 | | | 333.5 | |
| Total load | 18.5 | | | | | 2,800.5 | |
| Uplift | 5.3 | (9.16) | 8 | | | (388.33) | |
| Uplift | 10.3 | (9.04) | 8 | | | (744.76) | |
| Uplift | 2.9 | (8.94) | 8 | | | (207.33) | |
| Total (Load - Uplift) | | | | | | 1,460.0 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 353.44 |
| add. water pressure (Westergaard) | | | | | | | 1.18 |
| horizontal force from dead load (0.1g) | | | | | | | 280.05 |
| DS water pressure | ( water table at: | 64.00 m asl) | | | | | -225.00 |
| | | | | | | | 409.66 |

Friction angle (°) =  32  =  0.55851
tg  =  0.625

forces in flow direction:
US water pressure + Westergaard + dead load 0.1g:  634.66

forces inverse to flow direction:
DS water pressure  225.00
tg of load - uplift  1,460.0  * tg =  912.33
1,137.33

| **Safety Factor =** | 1,137.33 | / | 634.66 | = | **1.79** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case V:**     Normal - New 75 cm Slab, Headpond 65.90 m asl, tailwater 64.00 m asl

| | | | |
|---|---|---|---|
| | 64 | **61.00** m asl | **Water level:** |
| **Concrete Slab** | | Sand: | upstream:  **65.90**  m asl |
| | | spec. weight: | |
| | | **2.0** | downstr.:  **64.00**  m asl |
| **Sheet Pile Wall** | | t/m³ | |

vertical length at the end was considered with   **6.5**   weights

elevation:
**53.00**

$\underline{V}$

uplift force acting here =          (12.10)  t/m²

^ ^ ^ ^ ^ ^ ^

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00  t/m² |
| weight of add. water: | 3.00  t/m² |
| subtotal vertical forces | 19.00  t/m² |
| safety factor s =  vertical forces/uplift | 1.57 |
| Total vertical forces = | **6.90**  t/m² |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00  t/m² |
| remaining uplift from upstream: | (1.10)  t/m² |
| subtotal | 6.90  t/m² |
| **safety factor** (weight of sand/remain. uplift) **s =** | **7.26** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 1.90 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H =  **24.60** | > | **6 to 5** |
|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case VI:  Unusual - New 75 cm slab,**

**headpond 65.90 m asl, tailwater 60.50 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case VI:** Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 60.50 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 60.50 | m asl |
| D (H) = | 5.40 | m |
| Length of slab: | 51.70 | m |
| Bottom of sheet pile wall at the end of slab: | 53.00 | m asl |

vertical length at the end was considered with **6.5** weights ==> length of drainage = **100.45** m
Distance of DS Water Level to Bottom of Sheet Pile Wall **7.50** m

| | US end of Pier | | | Gate Section | | | | DS end of Pier | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 100.45 | 92.95 | 87.45 | 87.45 | 82.15 | 82.15 | 71.85 | 71.85 | 70.35 | 70.35 | 68.95 | 68.95 | 50.75 | 50.75 | 48.75 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | - | | |
| spec. weight (t/m³) | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 5.40 | 5.40 | 5.40 | 5.40 | 5.40 | - | - | - | - | - | - | - | - | - | - | |
| Total Weight (t/m²) | 9.88 | 11.03 | 11.03 | 14.56 | 14.56 | 9.16 | 9.16 | 9.08 | 9.08 | 10.88 | 10.88 | 8.00 | 8.00 | 6.20 | 6.20 | |
| water l. under found. for stability analysis | 65.90 | 65.50 | 65.20 | 65.20 | 64.92 | 64.92 | 64.36 | 64.36 | 64.28 | 64.28 | 64.21 | 64.21 | 63.23 | 63.23 | 63.12 | 60.50 |
| Uplift (t/m²) konst.p. | (2.00) | (2.50) | (2.50) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (4.00) | (2.75) | (2.75) | (2.75) | (2.75) | |
| Uplift (t/m²) linear p. | (5.40) | (5.00) | (4.70) | (4.70) | (4.42) | (4.42) | (3.86) | (3.86) | (3.78) | (3.78) | (3.71) | (3.71) | (2.73) | (2.73) | (2.62) | |
| Resulting Uplift | (7.40) | (7.50) | (7.20) | (8.70) | (8.42) | (8.42) | (7.86) | (7.86) | (7.78) | (7.78) | (7.71) | (6.46) | (5.48) | (5.48) | (5.37) | |
| Total | 2.48 | 3.53 | 3.82 | 5.86 | 6.14 | 0.74 | 1.30 | 1.21 | 1.29 | 3.09 | 3.17 | 1.54 | 2.52 | 0.72 | 0.83 | |
| Safety Factor s = | 1.33 | 1.47 | 1.53 | 1.67 | 1.73 | 1.09 | 1.17 | 1.15 | 1.17 | 1.40 | 1.41 | 1.24 | 1.46 | 1.13 | 1.15 | |

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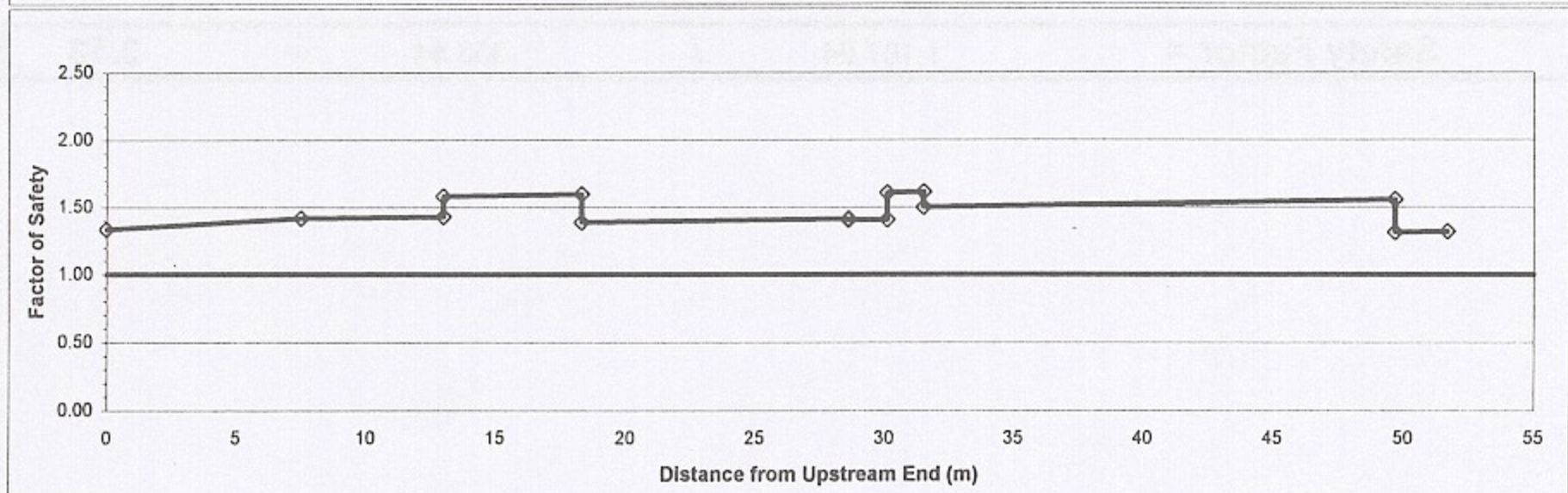Tender Design Report
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## Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case VI:**  Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 60.50 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 10.875 | 8 | | | 252.3 | |
| **Total load** | 18.5 | | | | | 2,383.3 | |
| Uplift | 5.3 | (8.56) | 8 | | | (362.89) | |
| Uplift | 10.3 | (8.14) | 8 | | | (670.68) | |
| Uplift | 2.9 | (7.78) | 8 | | | (180.60) | |
| **Total (Load - Uplift)** | | | | | | 1,169.2 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 353.44 |
| DS water pressure | ( water table at: | 60.50 m asl) | | | | | -64.00 |
| | | | | | | | 289.44 |

Friction angle (°) =  32  =  0.55851
          tg  =  0.625

forces in flow direction:
          US water pressure:                                353.44

forces inverse to flow direction:
          DS water pressure                                  64.00
          tg of load - uplift          1,169.2   * tg  =    730.57
                                                           794.57

| **Safety Factor =** | 794.57 | / | 353.44 | = | **2.25** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
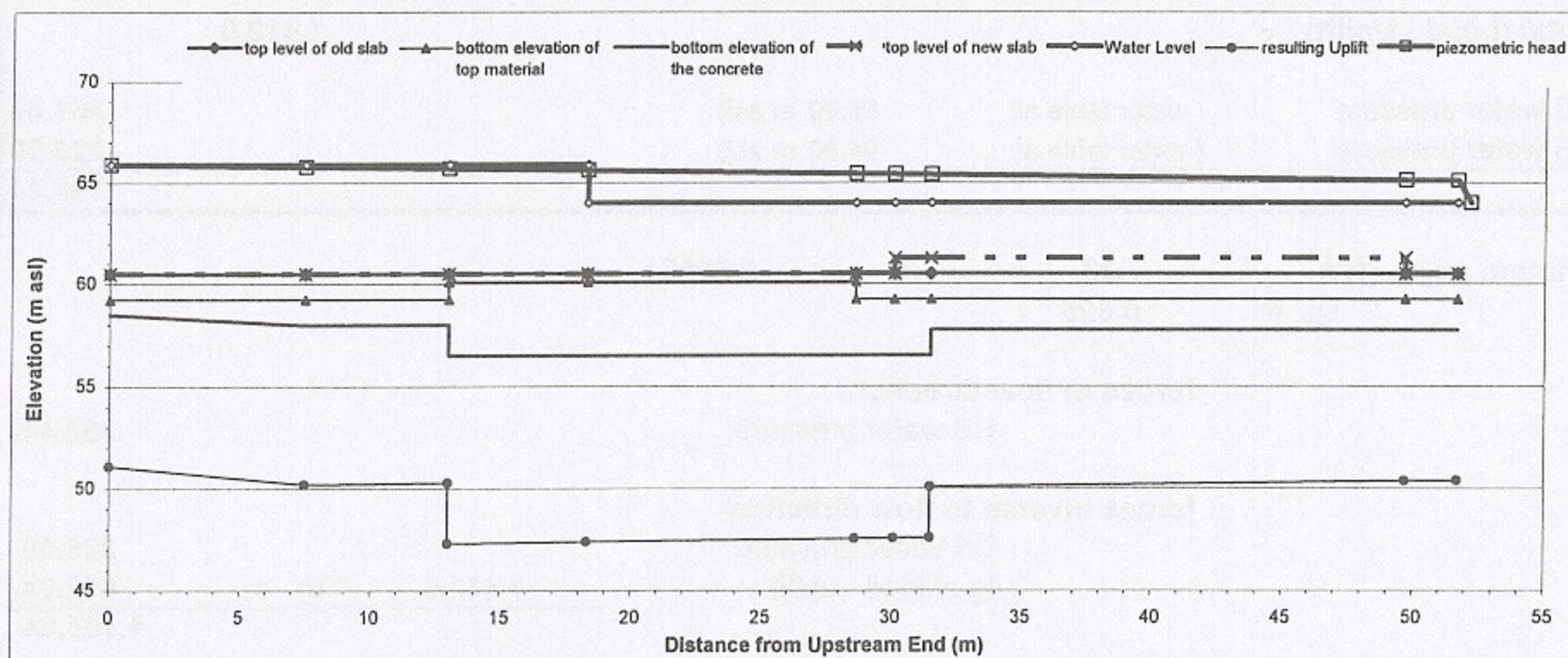## Foundation Slab of the Western Head Regulator

**Load Case VI:**     Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 60.50 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | height x spec.w. | | | | | | |
| found. slab | 5.3 | 14.56 | 8 | | | 617.3 | |
| found. slab | 10.3 | 9.16 | 8 | | | 754.8 | |
| found. slab | 2.9 | 10.875 | 8 | | | 252.3 | |
| Total load | 18.5 | | | | | 2,430.9 | |
| Uplift | 5.3 | (8.56) | 8 | | | (362.89) | |
| Uplift | 10.3 | (8.14) | 8 | | | (670.68) | |
| Uplift | 2.9 | (7.78) | 8 | | | (180.60) | |
| Total (Load - Uplift) | | | | | | 1,216.7 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 353.44 |
| add. water pressure (Westergaard) | | | | | | | 9.52 |
| horizontal force from dead load (0.1g) | | | | | | | 243.09 |
| DS water pressure | ( water table at: | | 60.50 m asl) | | | | -64.00 |
| | | | | | | | 542.04 |

Friction angle (°) =       32        =        0.55851
           tg  =     0.625

**forces in flow direction:**
US water pressure + Westergaard + dead load 0.1g:         606.04

**forces inverse to flow direction:**
DS water pressure                                          64.00
tg of load - uplift                 1,216.7   * tg  =     760.26
                                                          824.26

| **Safety Factor =** | 824.26 | / | 606.04 | = | **1.36** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case VI:**  Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 60.50 m asl

| **Concrete Slab** | elevation: | **61.00** m asl | **Water level:** | |
|---|---|---|---|---|
| | Sand: spec. weight: | | **upstream:** | **65.90** m asl |
| **Sheet Pile Wall** | **2.0** t/m$^3$ | | **downstr.:** | **60.50** m asl |

vertical length at the end was considered with **6.5** weights

elevation: **53.00**

$\underline{V}$

uplift force acting here = (10.12) t/m$^2$

∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 t/m$^2$ |
| weight of add. water: | -0.50 t/m$^2$ |
| subtotal vertical forces | 15.50 t/m$^2$ |
| safety factor s =  vertical forces/uplift | 1.53 |
| Total vertical forces = | **5.38** t/m$^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 t/m$^2$ |
| remaining uplift from upstream: | (2.62) t/m$^2$ |
| subtotal | 5.38 t/m$^2$ |

**safety factor** (weight of sand/remain. uplift) s = **3.05**

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 5.40 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | **8.65** | **>** | **6 to 5** |
|---|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :
| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case VII: Extreme - New 75 cm slab,**

**headpond 67.05 m asl, tailwater 64.00 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Western Head Regulator

**Load Case VII:** Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 67.05 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 3.05 | m |
| Length of slab: | 51.70 | m |
| Bottom of sheet pile wall at the end of slab: | 53.00 | m asl |

vertical length at the end was considered with **6.5** weights ==> length of drainage = **123.20** m
Distance of DS Water Level to Bottom of Sheet Pile Wall **11.00** m

| | | US end of Pier | | Gate Section | | | DS end of Pier | | | | Foundation Slab | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Distance from upstream end (m)** | - | 7.50 | 13.00 | 13.00 | 18.30 | 18.30 | 28.60 | 28.60 | 30.10 | 30.10 | 31.50 | 31.50 | 49.70 | 49.70 | 51.70 | 52.2 |
| Chainage | 123.20 | 115.70 | 110.20 | 110.20 | 104.90 | 104.90 | 94.60 | 94.60 | 93.10 | 93.10 | 91.70 | 91.70 | 73.50 | 73.50 | 71.50 | |
| top level of old slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | |
| top level of foundation s | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 60.5 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | 60.5 |
| bottom elevation of top material | 59.25 | 59.25 | 59.25 | 60.10 | 60.10 | 60.10 | 60.10 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | 59.25 | |
| bottom elevation of the concrete | 58.50 | 58.00 | 58.00 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 56.50 | 57.75 | 57.75 | 57.75 | 57.75 | |
| top level of new slab | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 60.50 | 61.25 | 61.25 | 61.25 | 61.25 | 60.50 | 60.50 | |
| thickness of top material (m) | 1.25 | 1.25 | 1.25 | 0.40 | 0.40 | 0.40 | 0.40 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | 1.25 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.75 | 1.25 | 1.25 | 3.60 | 3.60 | 3.60 | 3.60 | 2.75 | 2.75 | 2.75 | 2.75 | 1.50 | 1.50 | 1.50 | 1.50 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | - | | |
| spec. weight (t/m³) | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 6.55 | 6.55 | 6.55 | 6.55 | 6.55 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | 3.50 | |
| **Total Weight (t/m²)** | 11.03 | 12.18 | 12.18 | 15.71 | 15.71 | 12.66 | 12.66 | 12.58 | 12.58 | 14.38 | 14.38 | 11.50 | 11.50 | 9.70 | 9.70 | |
| water l. under found. for stability analysis | 67.05 | 66.86 | 66.73 | 66.73 | 66.60 | 66.60 | 66.34 | 66.34 | 66.30 | 66.30 | 66.27 | 66.27 | 65.82 | 65.82 | 65.77 | 64.00 |
| Uplift (t/m²) konst.p. | (5.50) | (6.00) | (6.00) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (7.50) | (6.25) | (6.25) | (6.25) | (6.25) | |
| Uplift (t/m²) linear p. | (3.05) | (2.86) | (2.73) | (2.73) | (2.60) | (2.60) | (2.34) | (2.34) | (2.30) | (2.30) | (2.27) | (2.27) | (1.82) | (1.82) | (1.77) | |
| **Resulting Uplift** | (8.55) | (8.86) | (8.73) | (10.23) | (10.10) | (10.10) | (9.84) | (9.84) | (9.80) | (9.80) | (9.77) | (8.52) | (8.07) | (8.07) | (8.02) | |
| **Total** | 2.48 | 3.31 | 3.45 | 5.48 | 5.61 | 2.56 | 2.82 | 2.73 | 2.77 | 4.57 | 4.60 | 2.98 | 3.43 | 1.63 | 1.68 | |
| **Safety Factor s =** | 1.29 | 1.37 | 1.39 | 1.54 | 1.56 | 1.25 | 1.29 | 1.28 | 1.28 | 1.47 | 1.47 | 1.35 | 1.43 | 1.20 | 1.21 | |

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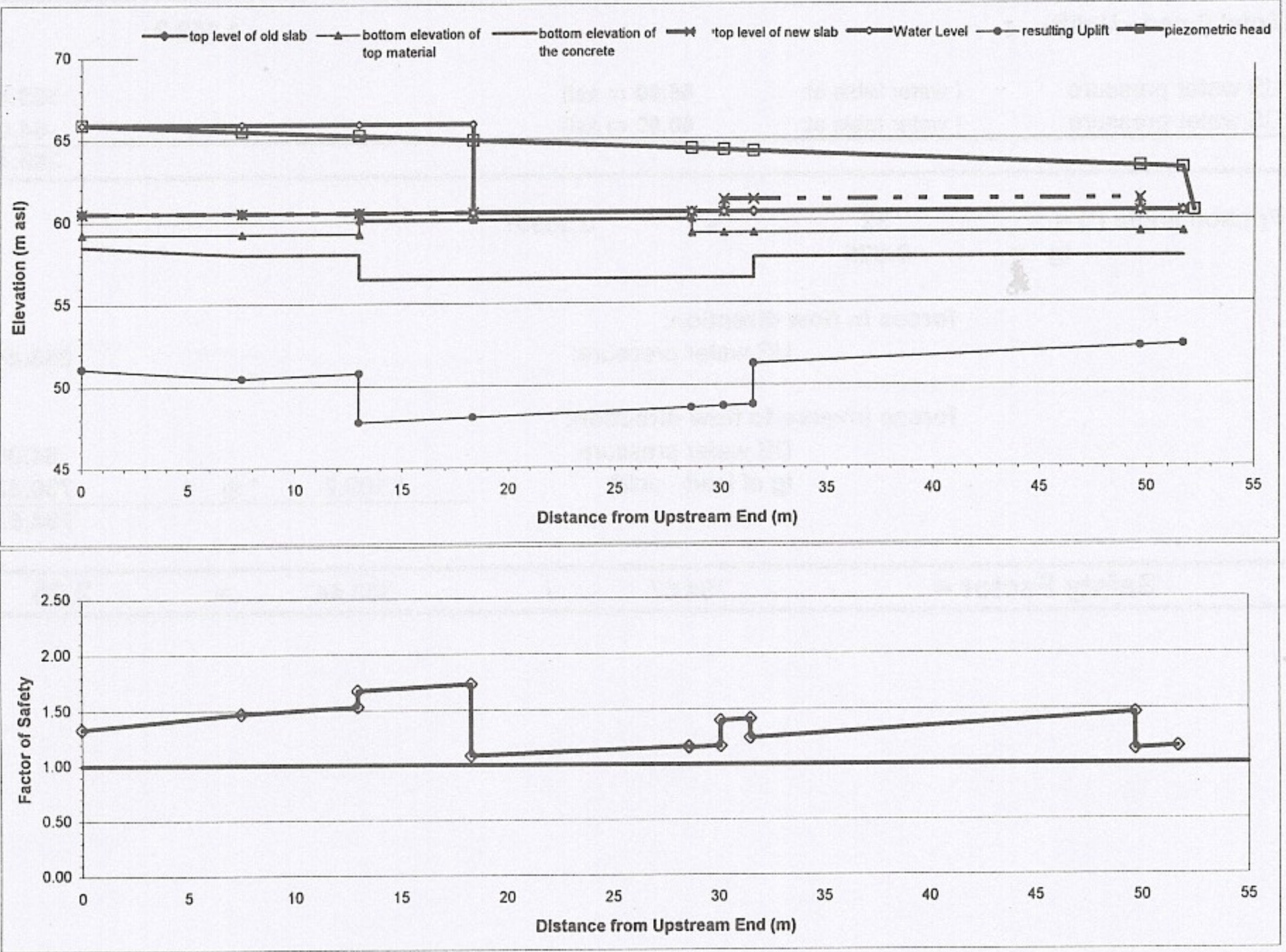Tender Design Report
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## Stability Analysis, Sliding
## Foundation Slab of the Western Head Regulator

**Load Case VII:**  Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m a

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | | 2.2 | - | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 345.0 | | 758.9 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 15.71 | 8 | | | 666.1 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 14.375 | 8 | | | 333.5 | |
| Total load | 18.5 | | | | | 2,801.7 | |
| Uplift | 5.3 | (10.16) | 8 | | | (430.89) | |
| Uplift | 10.3 | (9.97) | 8 | | | (821.48) | |
| Uplift | 2.9 | (9.81) | 8 | | | (227.50) | |
| Total (Load - Uplift) | | | | | | 1,321.8 | |
| US water pressure | ( water table at: | | 67.05 m asl) | | | | 445.21 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -225.00 |
| | | | | | | | 220.21 |

Friction angle (°) =  32  =  0.55851
tg  =  0.625

**forces in flow direction:**
US water pressure:  445.21

**forces inverse to flow direction:**
DS water pressure  225.00
tg of load - uplift  1,321.8  * tg =  825.96
1,050.96

| **Safety Factor =** | 1,050.96 | / | 445.21 | = | **2.36** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Western Head Regulator

**Load Case VII:**    Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m a

**Note:** from the foundation slab only the 4 m thick part is considered

|  | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.80 | 1.90 | 129.3 | 2.2 | 284.5 | |
| Total Pier | | | | 366.6 | | 806.4 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 15.71 | 8 | | | 666.1 | |
| found. slab | 10.3 | 12.66 | 8 | | | 1,043.2 | |
| found. slab | 2.9 | 14.375 | 8 | | | 333.5 | |
| Total load | 18.5 | | | | | 2,849.2 | |
| Uplift | 5.3 | (10.16) | 8 | | | (430.89) | |
| Uplift | 10.3 | (9.97) | 8 | | | (821.48) | |
| Uplift | 2.9 | (9.81) | 8 | | | (227.50) | |
| Total (Load - Uplift) | | | | | | 1,369.3 | |

| | | | | |
|---|---|---|---|---|
| US water pressure | ( water table at: | 67.05 m asl) | | 445.21 |
| add. water pressure (Westergaard) | | | | 3.04 |
| horizontal force from dead load (0.1g) | | | | 284.92 |
| DS water pressure | ( water table at: | 64.00 m asl) | | -225.00 |
| | | | | 508.17 |

Friction angle (°) =    32    =    0.55851

tg  =    0.625

**forces in flow direction:**
    US water pressure + Westergaard + dead load 0.1g:    733.17

**forces inverse to flow direction:**
    DS water pressure    225.00
    tg of load - uplift    1,369.3   * tg  =   855.66
    1,080.66

| **Safety Factor =** | 1,080.66 | / | 733.17 | = | **1.47** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Western Head Regulator

**Load Case VII:** Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m asl

67.05

64     **61.00** m asl     **Water level:**

**Concrete Slab**

Sand:     **upstream:**     **67.05** m asl
spec. weight:
**2.0**     **downstr.:**     **64.00** m asl
**Sheet Pile Wall**     $t/m^3$

vertical length at the end was considered with
**6.5**     weights

elevation:
**53.00**
V     uplift force acting here =     (12.77) $t/m^2$
∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 16.00 $t/m^2$ |
| weight of add. water: | 3.00 $t/m^2$ |
| subtotal vertical forces | 19.00 $t/m^2$ |
| safety factor s = vertical forces/uplift | 1.49 |
| Total vertical forces = | **6.23** $t/m^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 8.00 $t/m^2$ |
| remaining uplift from upstream: | (1.77) $t/m^2$ |
| subtotal | 6.23 $t/m^2$ |
| **safety factor** (weight of sand/remain. uplift) s = | **4.52** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 29.50 | =7+3.5+3.5+4.75+4.75+6 |
| horizontal distance along contact path Dh : | 51.70 | |
| total head loss H : | 3.05 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H =   **15.32**    >    **6 to 5** |
|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :
| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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# Foundation Slab of the Eastern Head Regulator

# Stability Analysis Eastern Head Regulator

## Summary

| No. | Load Case Water Level Upstream | Water Level Downstream | | Factor of Safety against Uplift at Downstream End | Sliding | Sliding with 10% Earthquake | Uplift 21.5 m from Downstrea | Uplift 13 m from Downstream | Uplift at Downstream End | Boiling | Piping (Lane) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **Existing structure without rehabilitation** | | | | | | | | | | | |
| **Normal Load Case** | | | | | | | | | | | |
| I | 65.10 | 64.00 | | 64.73 | 5.97 | 2.59 | 1.99 | 1.96 | 1.98 | 5.8 | 26.8 |
| II | 65.90 | 64.00 | | 65.27 | 4.72 | 2.27 | 1.83 | 1.78 | 1.81 | 3.4 | 15.5 |
| **Unusual Load Case** | | | | | | | | | | | |
| III | 65.10 | 61.25 | | 63.13 | 3.60 | 1.83 | 1.25 | **1.10** | 1.19 | 2.3 | 7.7 |
| IV | 65.90 | 61.25 | | 63.52 | 2.82 | 1.56 | 1.15 | **0.99** | **1.08** | 1.9 | 6.3 |
| **Rehabilitated structure with new additional 75 cm slab and increased headpond level** | | | | | | | | | | | |
| **Normal Load Case** | | | | | | | | | | | |
| V | 65.90 | 64.00 | | 65.27 | 4.78 | 2.29 | 2.07 | 2.06 | 2.10 | 3.4 | 15.5 |
| **Unusual Load Case** | | | | | | | | | | | |
| VI | 65.90 | 61.25 | | 63.52 | 2.88 | 1.58 | 1.44 | 1.36 | 1.48 | 1.9 | 6.3 |
| **Extreme Load Case** | | | | | | | | | | | |
| VII | 67.05 | 64.00 | | 66.03 | 3.52 | 1.90 | 1.85 | 1.82 | 1.86 | 2.1 | 9.7 |

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# Load Case I:  Normal - Existing structure

## headpond 65.10 m asl, tailwater level 64.00 m asl

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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case I:**  Normal - Existing structure, headpond 65.10 m asl, tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 65.10 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 1.10 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage **197.05** m

nce of DS Water Level to Bottom of Sheet Pile Wall  5.25  m

| | | US end of Pier | Gate Section | | | | DS end of Pier | | | | | Foundation Slab | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 65.3 |
| Chainage | 197.05 | 172.25 | 172.25 | 170.25 | 170.25 | 164.95 | 164.95 | 154.65 | 154.65 | 153.15 | 153.15 | 152.75 | 152.75 | 144.25 | 144.25 | 133.25 | 133.25 | 131.25 | |
| top level of old slab | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 |
| bottom elevation of top material | 63.00 | 64.00 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | |
| bottom elevation of the concrete | 62.50 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 5.65 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.00 | 4.25 | 4.05 | 4.05 | 4.05 | 4.05 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| spec. weight (t/m³) | | | | | | | | | | | | | | | | | | | |
| water load (t/m²) | 1.10 | 1.10 | 1.10 | 1.10 | 1.10 | 1.10 | - | - | - | - | - | - | - | - | - | - | - | - | |
| **Total Weight (t/m²)** | 4.45 | 14.10 | 15.44 | 15.44 | 15.44 | 15.44 | 14.34 | 14.34 | 14.34 | 14.34 | 14.34 | 14.34 | 13.37 | 11.65 | 11.19 | 11.19 | 11.19 | 11.19 | |
| water I. under found. for stability analysis | 65.10 | 64.96 | 64.96 | 64.95 | 64.95 | 64.92 | 64.92 | 64.86 | 64.86 | 64.85 | 64.85 | 64.85 | 64.85 | 64.81 | 64.81 | 64.74 | 64.74 | 64.73 | 64.00 |
| Uplift (t/m²) konst.p. | (1.50) | (5.65) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (5.85) | (5.10) | (4.90) | (4.90) | (4.90) | (4.90) | |
| Uplift (t/m²) linear p. | (1.10) | (0.96) | (0.96) | (0.95) | (0.95) | (0.92) | (0.92) | (0.86) | (0.86) | (0.85) | (0.85) | (0.85) | (0.85) | (0.81) | (0.81) | (0.74) | (0.74) | (0.73) | |
| **Resulting Uplift** | (2.60) | (6.61) | (7.21) | (7.20) | (7.20) | (7.17) | (7.17) | (7.11) | (7.11) | (7.10) | (7.10) | (7.10) | (6.70) | (5.91) | (5.71) | (5.64) | (5.64) | (5.63) | |
| **Total** | 1.85 | 7.48 | 8.22 | 8.23 | 8.23 | 8.26 | 7.16 | 7.22 | 7.22 | 7.23 | 7.23 | 7.23 | 6.67 | 5.74 | 5.48 | 5.54 | 5.54 | 5.55 | |
| **Safety Factor s =** | 1.71 | 2.13 | 2.14 | 2.14 | 2.14 | 2.15 | 2.00 | 2.02 | 2.02 | 2.02 | 2.02 | 2.02 | 1.99 | 1.97 | 1.96 | 1.98 | 1.98 | 1.99 | |

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## Stability Analysis, Sliding
### Foundation Slab of the Eastern Head Regulator

**Load Case I:**  Normal - Existing structure, headpond 65.10 m asl, tailwater 64.00 m as

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 15.435 | 8 | | | 654.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 14.335 | 8 | | | 217.9 | |
| Total load | 17.5 | | | | | 2,815.5 | |
| Uplift | 5.3 | (7.19) | 8 | | | (304.67) | |
| Uplift | 10.3 | (7.14) | 8 | | | (588.51) | |
| Uplift | 1.9 | (7.11) | 8 | | | (108.04) | |
| Total (Load - Uplift) | | | | | | 1,814.3 | |
| US water pressure | ( water table at: | | 65.10 m asl) | | | | 216.09 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 59.84 |

Friction angle (°) =     32     =     0.55851
       tg  =     0.625

forces in flow direction:
       US water pressure:     216.09

forces inverse to flow direction:
       DS water pressure     156.25
       tg of load - uplift     1,814.3     * tg  =     1,133.70
       1,289.95

| **Safety Factor =** | 1,289.95 | / | 216.09 | = | **5.97** |
|---|---|---|---|---|---|

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## Stability Analysis, Sliding with Earthquake of 0.1 g
### Foundation Slab of the Eastern Head Regulator

**Load Case I:**     Normal - Existing structure, headpond 65.10 m asl, tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 15.435 | 8 | | | 654.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 14.335 | 8 | | | 217.9 | |
| **Total load** | 17.5 | | | | | 2,815.5 | |
| Uplift | 5.3 | (7.19) | 8 | | | (304.67) | |
| Uplift | 10.3 | (7.14) | 8 | | | (588.51) | |
| Uplift | 1.9 | (7.11) | 8 | | | (108.04) | |
| **Total (Load - Uplift)** | | | | | | 1,814.3 | |
| US water pressure | ( water table at: | | 65.10 m asl) | | | | 216.09 |
| add. water pressure (Westergaard) | | | | | | | 0.39 |
| horizontal force from dead load (0.1g) | | | | | | | 281.55 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 341.79 |

Friction angle (°) =     32        =     0.55851
          tg  =     0.625

forces in flow direction:
          US water pressure:                                                 498.04

forces inverse to flow direction:
          DS water pressure                                           156.25
          tg of load - uplift                    1,814.3    * tg  =    1,133.70
                                                                          1,289.95

| **Safety Factor =** | 1,289.95 | / | 498.04 | = | **2.59** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Eastern Head Regulator

**Load Case I:**     Normal - Existing structure, headpond 65.10 m asl, tailwater 64.00 m asl

65.1

| | 64 | **63.00** m asl | **Water level:** |
|---|---|---|---|

**Concrete Slab**

Sand:     **upstream:**    **65.10**  m asl
spec. weight:
**2.0**     **downstr.:**    **64.00**  m asl
**t/m³**

**Sheet Pile Wall**

vertical length at the end was considered with
**25**     weights

elevation:
**58.75**

**V**     uplift force acting here =         **(5.98)** t/m²
∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 8.50 t/m² |
| weight of add. water: | 1.00 t/m² |
| subtotal vertical forces | 9.50 t/m² |
| safety factor s = vertical forces/uplift | 1.59 |
| Total vertical forces = | **3.52** t/m² |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 4.25 t/m² |
| remaining uplift from upstream: | (0.73) t/m² |
| subtotal | 3.52 t/m² |
| **safety factor** (weight of sand/remain. uplift) **s =** | **5.80** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)
vertical distance along contact path Dv :     7.50    =2+0.5+0.4+0.6+0.4+0.4+1.2+2
horizontal distance along contact path Dh :     65.80
total head loss H :     1.10

**Weighted Creep Ratio WCR =**     ( Dv + Dh / 3 ) / H =  **26.76**     **>**     **6 to 5**

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :
- Very fine sand or silt to fine sand     8.5 to 7
- **Medium to coarse sand**     **6 to 5**
- Fine to coarse gravel     4 to 3
- Boulders with some cobbles and gravel     2.5
- Soft to medium clay     3 to 2
- Hard to very hard clay     1.8 to 1.6

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**Load Case II:  Normal - Existing structure,
headpond 65.90 m asl, tailwater 64.00 m asl**

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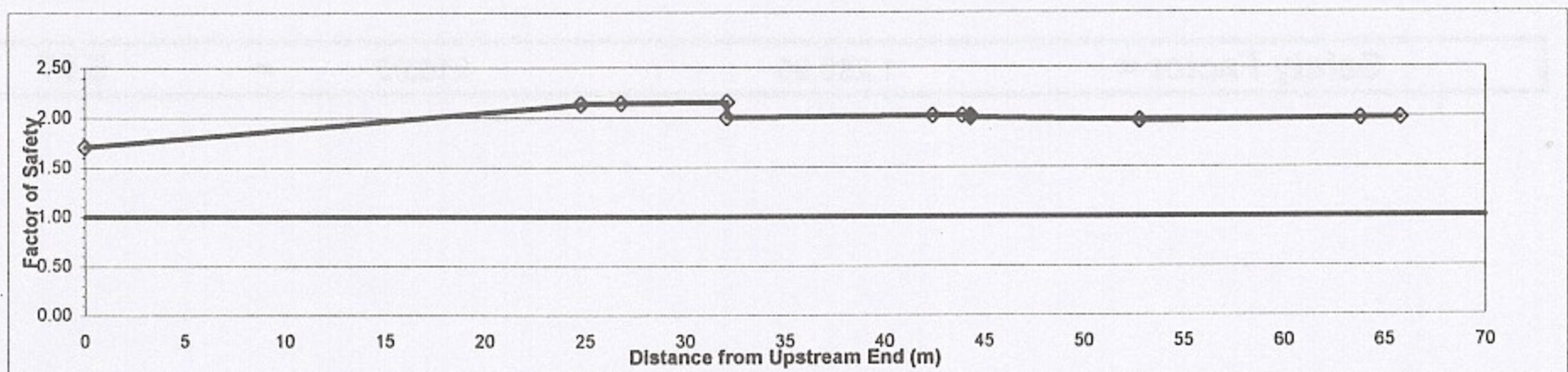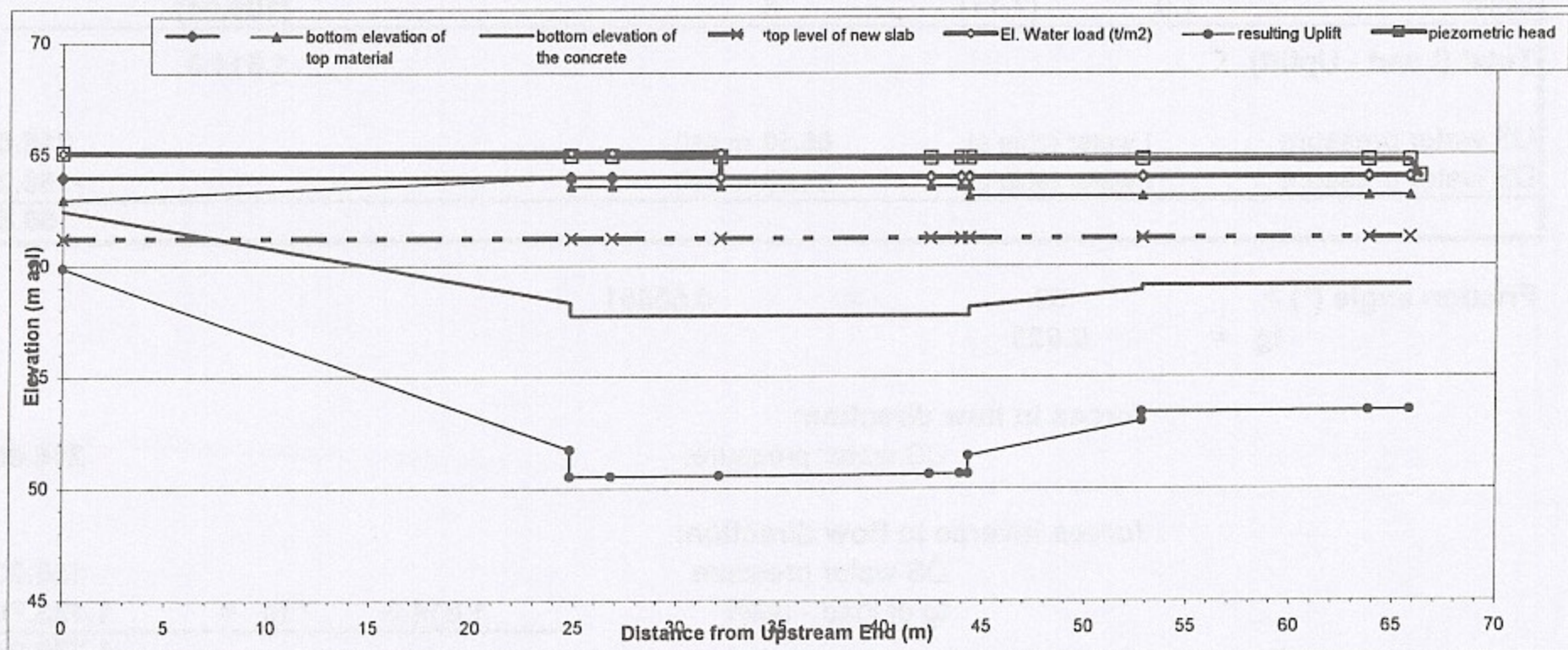Tender Design Report
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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case II:**   Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 1.90 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage **197.05** m

nce of DS Water Level to Bottom of Sheet Pile Wall  5.25  m

| | | US end of Pier | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 28.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 197.05 | 172.25 | 172.25 | 170.25 | 170.25 | 164.95 | 164.95 | 154.65 | 154.65 | 153.15 | 153.15 | 152.75 | 152.75 | 144.25 | 144.25 | 133.25 | 133.25 | 131.25 | |
| top level of old slab | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 |
| bottom elevation of top material | 63.00 | 64.00 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | |
| bottom elevation of the concrete | 62.50 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 5.65 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.00 | 4.25 | 4.05 | 4.05 | 4.05 | 4.05 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | | |
| spec. weight (t/m³) | | | | | | | | | | | | | | | | | | | |
| water load (t/m²) | 1.90 | 1.90 | 1.90 | 1.90 | 1.90 | 1.90 | - | - | - | - | - | - | - | - | - | - | - | - | |
| **Total Weight (t/m²)** | 5.25 | 14.90 | 16.24 | 16.24 | 16.24 | 16.24 | 14.34 | 14.34 | 14.34 | 14.34 | 14.34 | 14.34 | 13.37 | 11.65 | 11.19 | 11.19 | 11.19 | 11.19 | |
| water l. under found. for stability analysis | 65.90 | 65.66 | 65.66 | 65.64 | 65.64 | 65.59 | 65.59 | 65.49 | 65.49 | 65.48 | 65.48 | 65.47 | 65.47 | 65.39 | 65.39 | 65.28 | 65.28 | 65.27 | 64.00 |
| Uplift (t/m²) konst.p. | (1.50) | (5.65) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (5.85) | (5.10) | (4.90) | (4.90) | (4.90) | (4.90) | |
| Uplift (t/m²) linear p. | (1.90) | (1.66) | (1.66) | (1.64) | (1.64) | (1.59) | (1.59) | (1.49) | (1.49) | (1.48) | (1.48) | (1.47) | (1.47) | (1.39) | (1.39) | (1.28) | (1.28) | (1.27) | |
| **Resulting Uplift** | (3.40) | (7.31) | (7.91) | (7.89) | (7.89) | (7.84) | (7.84) | (7.74) | (7.74) | (7.73) | (7.73) | (7.72) | (7.32) | (6.49) | (6.29) | (6.18) | (6.18) | (6.17) | |
| **Total** | 1.85 | 7.58 | 8.32 | 8.34 | 8.34 | 8.39 | 6.49 | 6.59 | 6.59 | 6.61 | 6.61 | 6.61 | 6.05 | 5.15 | 4.89 | 5.00 | 5.00 | 5.02 | |
| **Safety Factor s =** | 1.54 | 2.04 | 2.05 | 2.06 | 2.06 | 2.07 | 1.83 | 1.85 | 1.85 | 1.86 | 1.86 | 1.86 | 1.83 | 1.79 | 1.78 | 1.81 | 1.81 | 1.81 | |

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## Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case II:**   Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 16.235 | 8 | | | 688.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 14.335 | 8 | | | 217.9 | |
| **Total load** | 17.5 | | | | | 2,849.4 | |
| Uplift | 5.3 | (7.87) | 8 | | | (333.52) | |
| Uplift | 10.3 | (7.79) | 8 | | | (641.96) | |
| Uplift | 1.9 | (7.73) | 8 | | | (117.53) | |
| Total (Load - Uplift) | | | | | | 1,756.4 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 265.69 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 109.44 |

Friction angle (°) =    32    =    0.55851
        tg  =    0.625

forces in flow direction:
  US water pressure:                                                                 265.69

forces inverse to flow direction:
  DS water pressure                                                                  156.25
  tg of load - uplift          1,756.4      * tg  =          1,097.53
                                                                                    1,253.78

| **Safety Factor =** | 1,253.78 | / | 265.69 | = | **4.72** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case II:**     Normal - Existing structure, headpond 65.90 m asl, tailwater 64.00 m as

**Note:** from the foundation slab only the 4 m thick part is considered

|  | length (in flow dir.) | 64 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| **Total Pier** | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 16.235 | 8 | | | 688.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 14.335 | 8 | | | 217.9 | |
| **Total load** | 17.5 | | | | | **2,849.4** | |
| Uplift | 5.3 | (7.87) | 8 | | | (333.52) | |
| Uplift | 10.3 | (7.79) | 8 | | | (641.96) | |
| Uplift | 1.9 | (7.73) | 8 | | | (117.53) | |
| **Total (Load - Uplift)** | | | | | | **1,756.4** | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 265.69 |
| add. water pressure (Westergaard) | | | | | | | 1.18 |
| horizontal force from dead load (0.1g) | | | | | | | 284.94 |
| DS water pressure | ( water table at: | 64.00 m asl) | | | | | -156.25 |
| | | | | | | | 395.56 |

Friction angle (°) =     32     =     0.55851
tg  =     0.625

forces in flow direction:
        US water pressure:                                                      551.81

forces inverse to flow direction:
        DS water pressure                                                       156.25
        tg of load - uplift          1,756.4     * tg  =          1,097.53
                                                                                          1,253.78

| **Safety Factor =** | 1,253.78 | / | 551.81 | = | **2.27** |
|---|---|---|---|---|---|

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# Load Case III:  Unusual - Existing structure
# with low tailwater level at 61.25 m asl

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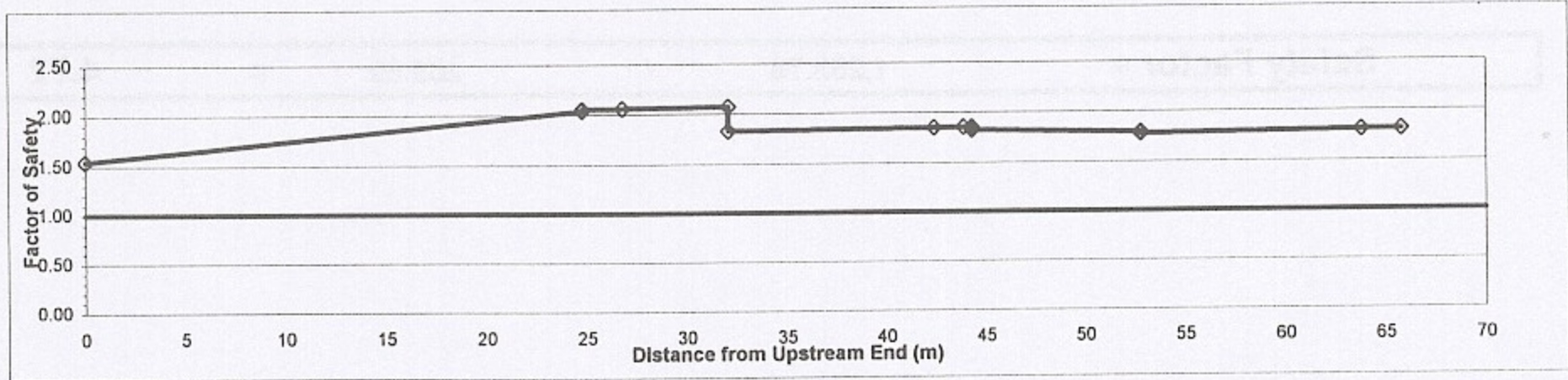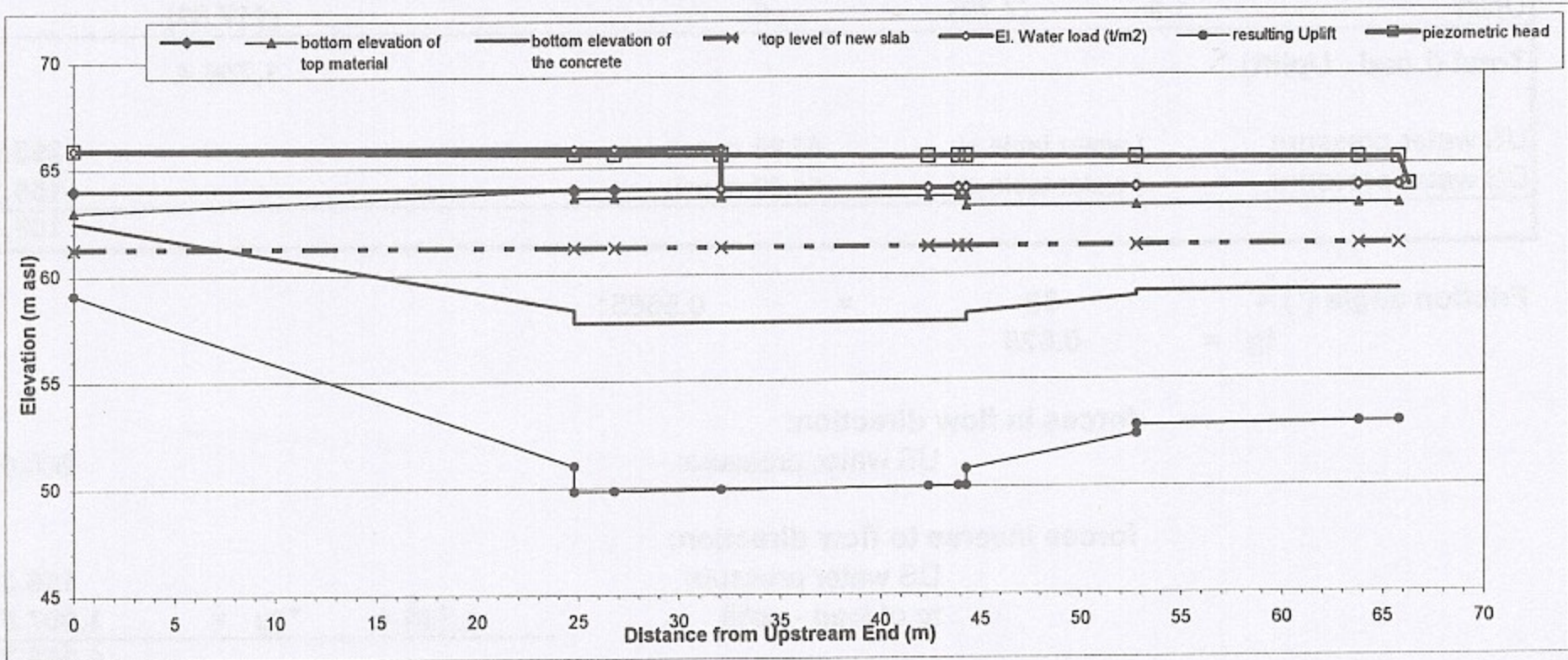
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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case III:** Unusual - Existing structure with low tailwater level at 61.25 m asl

| | | |
|---|---|---|
| Water level US: | 65.10 | m asl |
| Water level DS: | 61.25 | m asl |
| D (H) = | 3.85 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage **128.30** m

Distance of DS Water Level to Bottom of Sheet Pile Wall 2.50 m

| | | US end of Pier | | | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 128.30 | 103.50 | 103.50 | 101.50 | 101.50 | 96.20 | 96.20 | 85.90 | 85.90 | 84.40 | 84.40 | 84.00 | 84.00 | 75.50 | 75.50 | 64.50 | 64.50 | 62.50 | |
| top level of old slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| bottom elevation of top material | 60.25 | 61.25 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | |
| bottom elevation of the concrete | 59.75 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 2.90 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 2.25 | 1.50 | 1.30 | 1.30 | 1.30 | 1.30 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | |
| spec. weight (t/m³) | | | | | | | | | | | | | | | | | | | |
| water load (t/m²) | 3.85 | 3.85 | 3.85 | 3.85 | 3.85 | 3.85 | - | - | - | - | - | - | - | - | - | - | - | - | |
| Total Weight (t/m²) | 7.20 | 10.52 | 11.86 | 11.86 | 11.86 | 11.86 | 8.01 | 8.01 | 8.01 | 8.01 | 8.01 | 8.01 | 7.05 | 5.32 | 4.86 | 4.86 | 4.86 | 4.86 | |
| water l. under found. for stability analysis | 65.10 | 64.36 | 64.36 | 64.30 | 64.30 | 64.14 | 64.14 | 63.83 | 63.83 | 63.78 | 63.78 | 63.77 | 63.77 | 63.52 | 63.52 | 63.19 | 63.19 | 63.13 | 61.25 |
| Uplift (t/m²) konst.p. | (1.50) | (2.90) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.10) | (2.35) | (2.15) | (2.15) | (2.15) | (2.15) | |
| Uplift (t/m²) linear p. | (3.85) | (3.11) | (3.11) | (3.05) | (3.05) | (2.89) | (2.89) | (2.58) | (2.58) | (2.53) | (2.53) | (2.52) | (2.52) | (2.27) | (2.27) | (1.94) | (1.94) | (1.88) | |
| Resulting Uplift | (5.35) | (6.01) | (6.61) | (6.55) | (6.55) | (6.39) | (6.39) | (6.08) | (6.08) | (6.03) | (6.03) | (6.02) | (5.62) | (4.62) | (4.42) | (4.09) | (4.09) | (4.03) | |
| Total | 1.85 | 4.51 | 5.25 | 5.31 | 5.31 | 5.47 | 1.62 | 1.93 | 1.93 | 1.98 | 1.98 | 1.99 | 1.42 | 0.70 | 0.44 | 0.77 | 0.77 | 0.83 | |
| Safety Factor s = | 1.35 | 1.75 | 1.80 | 1.81 | 1.81 | 1.86 | 1.25 | 1.32 | 1.32 | 1.33 | 1.33 | 1.33 | 1.25 | 1.15 | 1.10 | 1.19 | 1.19 | 1.21 | |

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# Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case III:**     Unusual - Existing structure with low tailwater level at 61.25 m asl

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 61.25 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 11.86 | 8 | | | 502.9 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 8.01 | 8 | | | 121.8 | |
| Total load | 17.5 | | | | | 2,046.6 | |
| Uplift | 5.3 | (6.47) | 8 | | | (274.17) | |
| Uplift | 10.3 | (6.23) | 8 | | | (513.53) | |
| Uplift | 1.9 | (6.05) | 8 | | | (91.95) | |
| Total (Load - Uplift) | | | | | | 1,167.0 | |
| US water pressure | ( water table at: | 65.10 m asl) | | | | | 216.09 |
| DS water pressure | ( water table at: | 61.25 m asl) | | | | | -49.00 |
| | | | | | | | 167.09 |

Friction angle (°) =     32     =     0.55851
            tg  =     0.625

forces in flow direction:
        US water pressure:                                   216.09

forces inverse to flow direction:
        DS water pressure                                    49.00
        tg of load - uplift          1,167.0     * tg  =     729.20
                                                             778.20

| **Safety Factor =** | 778.20 | / | 216.09 | = | **3.60** |
|---|---|---|---|---|---|

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## Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case III:**     Unusual - Existing structure with low tailwater level at 61.25 m asl

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 61.25 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 11.86 | 8 | | | 502.9 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 8.01 | 8 | | | 121.8 | |
| Total load | 17.5 | | | | | 2,046.6 | |
| Uplift | 5.3 | (6.47) | 8 | | | (274.17) | |
| Uplift | 10.3 | (6.23) | 8 | | | (513.53) | |
| Uplift | 1.9 | (6.05) | 8 | | | (91.95) | |
| Total (Load - Uplift) | | | | | | 1,167.0 | |
| US water pressure | ( water table at: | 65.10 m asl) | | | | | 216.09 |
| add. water pressure (Westergaard) | | | | | | | 4.84 |
| horizontal force from dead load (0.1g) | | | | | | | 204.66 |
| DS water pressure | ( water table at: | 61.25 m asl) | | | | | -49.00 |
| | | | | | | | 376.59 |

Friction angle (°) =    32     =     0.55851

tg   =    0.625

forces in flow direction:
     US water pressure:                    425.59

forces inverse to flow direction:
     DS water pressure                        49.00

     tg of load - uplift      1,167.0    * tg =    729.20

                                               778.20

| **Safety Factor =** | 778.20 | / | 425.59 | = | **1.83** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
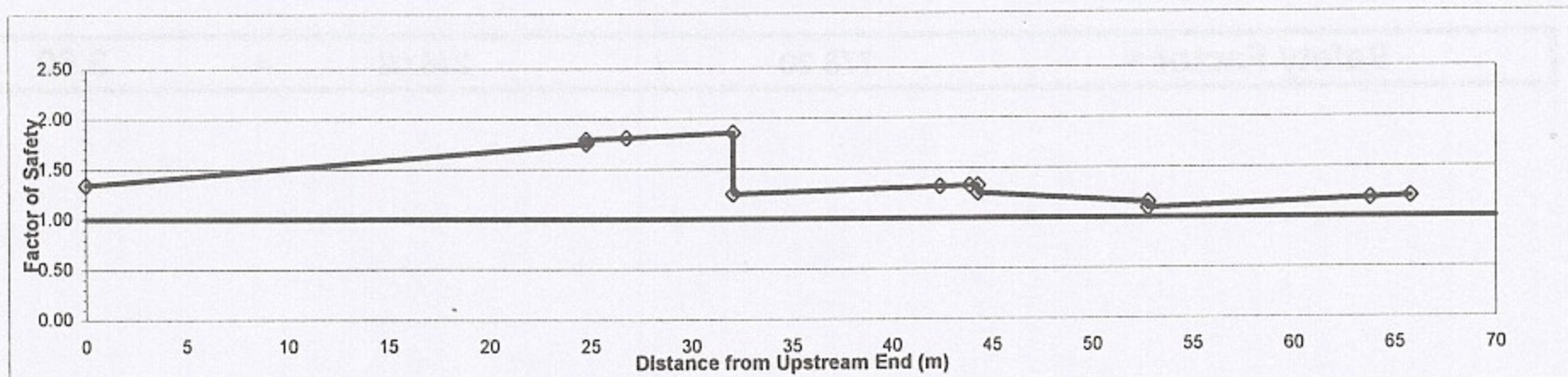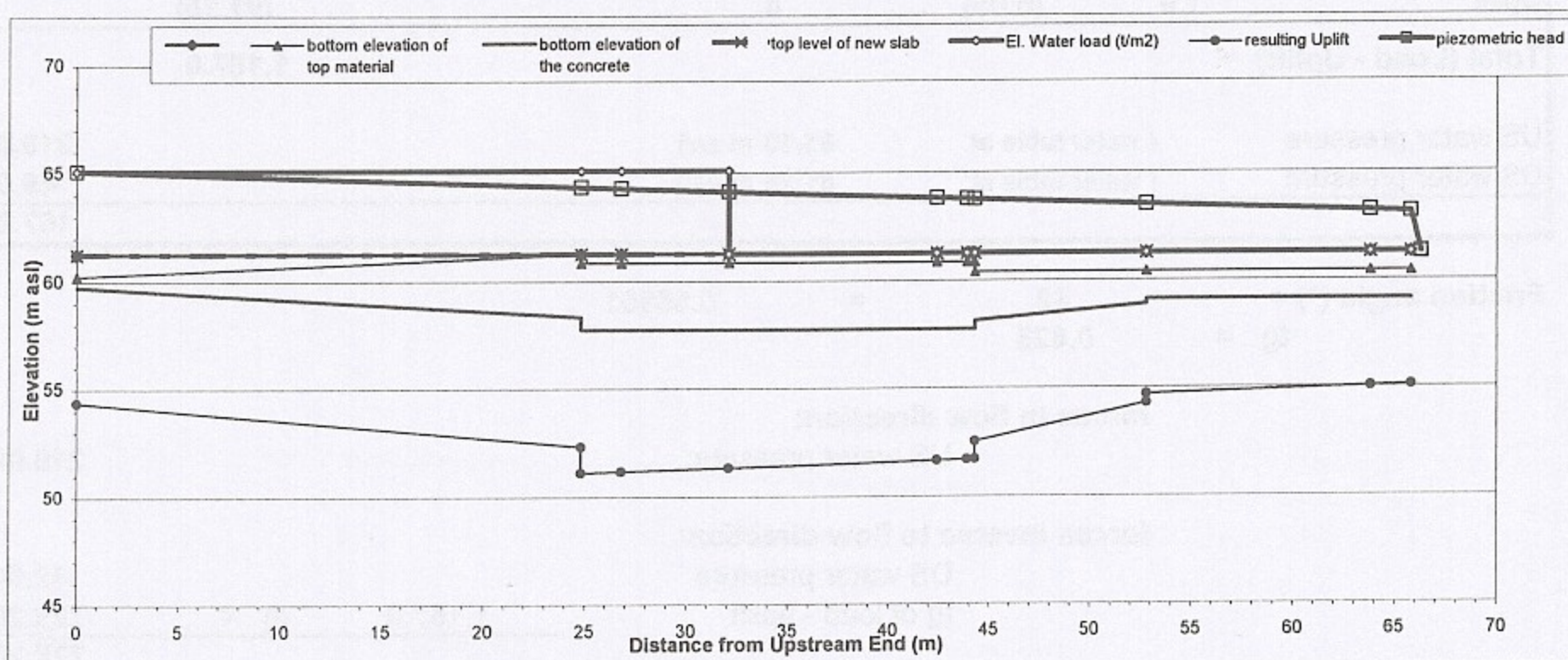## Foundation Slab of the Eastern Head Regulator

**Load Case III:**    Unusual - Existing structure with low tailwater level at 61.25 m asl

65.1

61.25    **63.00**   m asl    **Water level:**

**Concrete Slab**

Sand:      **upstream:**    **65.10**   m asl

spec. weight:

**2.0**      **downstr.:**    **61.25**   m asl

**Sheet Pile Wall**   $t/m^3$

vertical length at the end was considered with

**25**    weights

elevation:

**58.75**

V     uplift force acting here =      (4.38) $t/m^2$

^ ^ ^ ^ ^ ^ ^

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 8.50 $t/m^2$ |
| weight of add. water: | -1.75 $t/m^2$ |
| subtotal vertical forces | 6.75 $t/m^2$ |
| safety factor s = vertical forces/uplift | 1.54 |
| Total vertical forces = | **2.37** $t/m^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 4.25 $t/m^2$ |
| remaining uplift from upstream: | (1.88) $t/m^2$ |
| subtotal | 2.37 $t/m^2$ |
| **safety factor** (weight of sand/remain. uplift) **s =** | **2.27** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 7.50 | =2+0.5+0.4+0.6+0.4+0.4+1.2+2 |
| horizontal distance along contact path Dh : | 65.80 | |
| total head loss H : | 3.85 | |

| | | | | |
|---|---|---|---|---|
| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | **7.65** | **>** | **6 to 5** |

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case IV:  Unusual - Existing structure,
headpond 65.90 m asl, tailwater 61.25 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case IV:**   Unusual - Existing Structure, Headpond 65.90 m asl, Tailwater 61.25 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 61.25 | m asl |
| D (H) = | 4.65 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage **128.30** m

nce of DS Water Level to Bottom of Sheet Pile Wall   2.50   m

| | | US end of Pier | Gate Section | | DS end of Pier | | | | | Foundation Slab | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 128.30 | 103.50 | 103.50 | 101.50 | 101.50 | 96.20 | 96.20 | 85.90 | 85.90 | 84.40 | 84.40 | 84.00 | 84.00 | 75.50 | 75.50 | 64.50 | 64.50 | 62.50 | |
| top level of old slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 |
| bottom elevation of top material | 60.25 | 61.25 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | |
| bottom elevation of the concrete | 59.75 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 2.90 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 2.25 | 1.50 | 1.30 | 1.30 | 1.30 | 1.30 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | |
| spec. weight (t/m³) | | | | | | | | | | | | | | | | | | | |
| water load (t/m²) | 4.65 | 4.65 | 4.65 | 4.65 | 4.65 | 4.65 | - | - | - | - | - | - | - | - | - | - | - | - | |
| **Total Weight (t/m²)** | 8.00 | 11.32 | 12.66 | 12.66 | 12.66 | 12.66 | 8.01 | 8.01 | 8.01 | 8.01 | 8.01 | 8.01 | 7.05 | 5.32 | 4.86 | 4.86 | 4.86 | 4.86 | |
| water l. under found. for stability analysis | 65.90 | 65.00 | 65.00 | 64.93 | 64.93 | 64.74 | 64.74 | 64.36 | 64.36 | 64.31 | 64.31 | 64.29 | 64.29 | 63.99 | 63.99 | 63.59 | 63.59 | 63.52 | 61.25 |
| Uplift (t/m²) konst.p. | (1.50) | (2.90) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.10) | (2.35) | (2.15) | (2.15) | (2.15) | (2.15) | |
| Uplift (t/m²) linear p. | (4.65) | (3.75) | (3.75) | (3.68) | (3.68) | (3.49) | (3.49) | (3.11) | (3.11) | (3.06) | (3.06) | (3.04) | (3.04) | (2.74) | (2.74) | (2.34) | (2.34) | (2.27) | |
| **Resulting Uplift** | (6.15) | (6.65) | (7.25) | (7.18) | (7.18) | (6.99) | (6.99) | (6.61) | (6.61) | (6.56) | (6.56) | (6.54) | (6.14) | (5.09) | (4.89) | (4.49) | (4.49) | (4.42) | |
| **Total** | 1.85 | 4.67 | 5.41 | 5.48 | 5.48 | 5.67 | 1.02 | 1.40 | 1.40 | 1.45 | 1.45 | 1.47 | 0.90 | 0.23 | (0.03) | 0.37 | 0.37 | 0.44 | |
| **Safety Factor s =** | 1.30 | 1.70 | 1.75 | 1.76 | 1.76 | 1.81 | 1.15 | 1.21 | 1.21 | 1.22 | 1.22 | 1.22 | 1.15 | 1.05 | 0.99 | 1.08 | 1.08 | 1.10 | |

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# Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case IV:**   Unusual - Existing Structure, Headpond 65.90 m asl, Tailwater 61.25 m

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | 61.25 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 12.66 | 8 | | | 536.8 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 8.01 | 8 | | | 121.8 | |
| **Total load** | 17.5 | | | | | **2,080.5** | |
| Uplift | 5.3 | (7.08) | 8 | | | (300.30) | |
| Uplift | 10.3 | (6.80) | 8 | | | (560.32) | |
| Uplift | 1.9 | (6.58) | 8 | | | (100.00) | |
| **Total (Load - Uplift)** | | | | | | 1,119.9 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 265.69 |
| DS water pressure | ( water table at: | 61.25 m asl) | | | | | -49.00 |
| | | | | | | | 216.69 |

Friction angle (°) =   32   =   0.55851
tg  =   0.625

forces in flow direction:
US water pressure:                                                                     265.69

forces inverse to flow direction:
DS water pressure                                                                      49.00
tg of load - uplift            1,119.9      * tg  =      699.80
                                                                                        748.80

| **Safety Factor =** | 748.80 | / | 265.69 | = | **2.82** |
|---|---|---|---|---|---|

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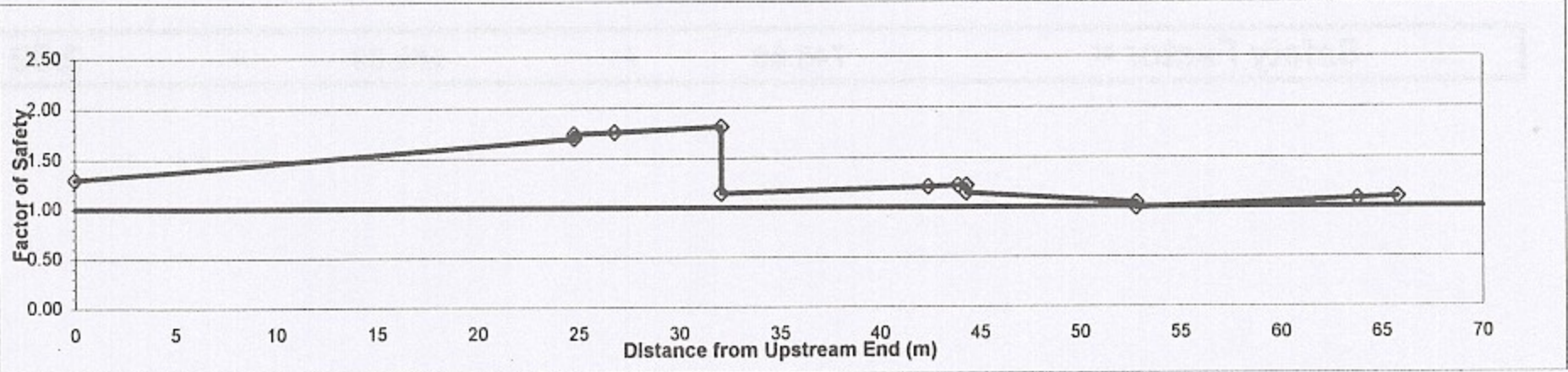September 2000

# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case IV:**     Unusual - Existing Structure, Headpond 65.90 m asl, Tailwater 61.25 m

**Note:** from the foundation slab only the 4 m thick part is considered

|  | length (in flow dir.) | 61.25 m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 12.66 | 8 | | | 536.8 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 8.01 | 8 | | | 121.8 | |
| Total load | 17.5 | | | | | 2,080.5 | |
| Uplift | 5.3 | (7.08) | 8 | | | (300.30) | |
| Uplift | 10.3 | (6.80) | 8 | | | (560.32) | |
| Uplift | 1.9 | (6.58) | 8 | | | (100.00) | |
| Total (Load - Uplift) | | | | | | 1,119.9 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 265.69 |
| add. water pressure (Westergaard) | | | | | | | 7.06 |
| horizontal force from dead load (0.1g) | | | | | | | 208.05 |
| DS water pressure | ( water table at: | 61.25 m asl) | | | | | -49.00 |
| | | | | | | | 431.80 |

Friction angle (°) =    32    =    0.55851
     tg =    0.625

forces in flow direction:
     US water pressure:          480.80

forces inverse to flow direction:
     DS water pressure          49.00
     tg of load - uplift     1,119.9   * tg =    699.80
                      748.80

| **Safety Factor =** | 748.80 | / | 480.80 | = | **1.56** |
|---|---|---|---|---|---|

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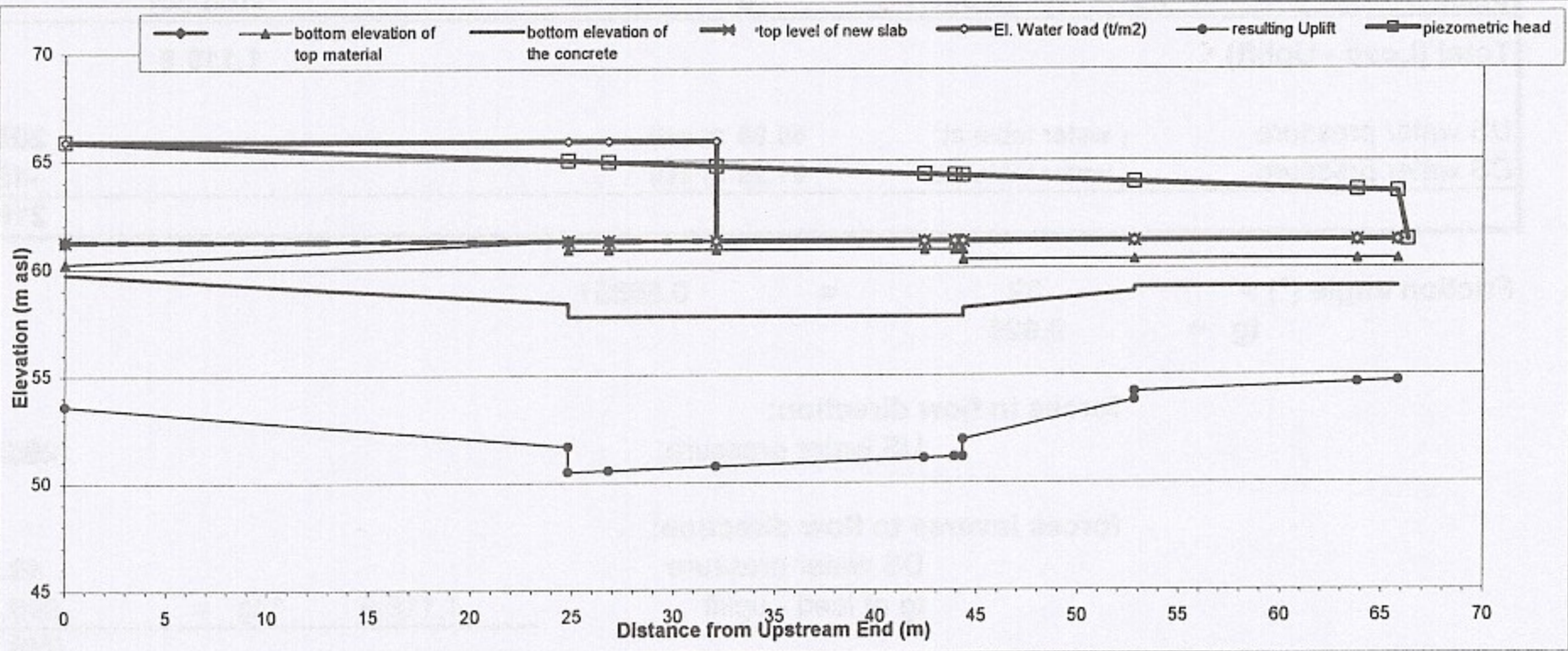Volume 1
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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Eastern Head Regulator

**Load Case IV:**  Unusual - Existing Structure, Headpond 65.90 m asl, Tailwater 61.25 m asl

65.9

61.25   **63.00**   m asl   **Water level:**

**Concrete Slab**

Sand:   **upstream:**   **65.90**  m asl
spec. weight:

**2.0**   **downstr.:**   **61.25**  m asl
**Sheet Pile Wall**   t/m³

vertical length at the end was considered with
**25**   weights

elevation:

**58.75**

V   uplift force acting here =   (4.77) t/m²

∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 8.50  t/m² |
| weight of add. water: | -1.75  t/m² |
| subtotal vertical forces | 6.75  t/m² |
| safety factor s =  vertical forces/uplift | 1.42 |
| Total vertical forces = | **1.98**  t/m² |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 4.25  t/m² |
| remaining uplift from upstream: | (2.27)  t/m² |
| subtotal | 1.98  t/m² |
| **safety factor** (weight of sand/remain. uplift) s = | **1.88** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 7.50 | =2+0.5+0.4+0.6+0.4+0.4+1.2+2 |
| horizontal distance along contact path Dh : | 65.80 | |
| total head loss H : | 4.65 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | 6.33 | > | 6 to 5 |
|---|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case V:  Normal - New 75 cm slab,
headpond 65.90 m asl, tailwater 64.00 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case V:** Normal - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 1.90 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with 25 weights ==> length of drainage 197.05 m
nce of DS Water Level to Bottom of Sheet Pile Wall 5.25 m

| | | US end of Pier | | Gate Section | | | DS end of Pier | | | | | Foundation Slab | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 197.05 | 172.25 | 172.25 | 170.25 | 170.25 | 164.95 | 164.95 | 154.65 | 154.65 | 153.15 | 153.15 | 152.75 | 152.75 | 144.25 | 144.25 | 133.25 | 133.25 | 131.25 |
| top level of old slab | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 | #### |
| bottom elevation of top material | 63.00 | 64.00 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 |
| bottom elevation of the concrete | 62.50 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 |
| thickness of the concrete (m) | 0.50 | 5.65 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.00 | 4.25 | 4.05 | 4.05 | 4.05 | 4.05 |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 |
| thickness of new concrete (m) | | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | | |
| spec. weight (t/m³) | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | | |
| water load (t/m²) | 1.90 | 1.90 | 1.90 | 1.90 | 1.90 | 1.90 | - | - | - | - | - | - | - | - | - | - | - | - |
| **Total Weight (t/m²)** | 5.25 | 14.90 | 16.24 | 16.24 | 16.24 | 16.24 | 14.34 | 14.34 | 14.34 | 14.34 | 16.14 | 16.14 | 15.17 | 13.45 | 12.99 | 12.99 | 11.19 | 11.19 |
| water l. under found. for stability analysis | 65.90 | 65.66 | 65.66 | 65.64 | 65.64 | 65.59 | 65.59 | 65.49 | 65.49 | 65.48 | 65.48 | 65.47 | 65.47 | 65.39 | 65.39 | 65.28 | 65.28 | 65.27 | #### |
| Uplift (t/m²) konst.p. | (1.50) | (5.65) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (5.85) | (5.10) | (4.90) | (4.90) | (4.90) | (4.90) |
| Uplift (t/m²) linear p. | (1.90) | (1.66) | (1.66) | (1.64) | (1.64) | (1.59) | (1.59) | (1.49) | (1.49) | (1.48) | (1.48) | (1.47) | (1.47) | (1.39) | (1.39) | (1.28) | (1.28) | (1.27) |
| **Resulting Uplift** | (3.40) | (7.31) | (7.91) | (7.89) | (7.89) | (7.84) | (7.84) | (7.74) | (7.74) | (7.73) | (7.73) | (7.72) | (7.32) | (6.49) | (6.29) | (6.18) | (6.18) | (6.17) |
| **Total** | 1.85 | 7.58 | 8.32 | 8.34 | 8.34 | 8.39 | 6.49 | 6.59 | 6.59 | 6.61 | 8.41 | 8.41 | 7.85 | 6.95 | 6.69 | 6.80 | 5.00 | 5.02 |
| **Safety Factor s =** | 1.54 | 2.04 | 2.05 | 2.06 | 2.06 | 2.07 | 1.83 | 1.85 | 1.85 | 1.86 | 2.09 | 2.09 | 2.07 | 2.07 | 2.06 | 2.10 | 1.81 | 1.81 |

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# Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case V:**     Normal - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 64.00 m as

**Note:** from the foundation slab only the 4 m thick part is considered

|  | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 16.235 | 8 | | | 688.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 16.135 | 8 | | | 245.3 | |
| **Total load** | 17.5 | | | | | 2,876.8 | |
| Uplift | 5.3 | (7.87) | 8 | | | (333.52) | |
| Uplift | 10.3 | (7.79) | 8 | | | (641.96) | |
| Uplift | 1.9 | (7.73) | 8 | | | (117.53) | |
| **Total (Load - Uplift)** | | | | | | 1,783.8 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 265.69 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 109.44 |

Friction angle (°) =     32          =      0.55851
          tg   =      0.625

forces in flow direction:
      US water pressure:                                                    265.69

forces inverse to flow direction:
      DS water pressure                                                     156.25
      tg of load - uplift          1,783.8     * tg  =          1,114.63
                                                                           1,270.88

| **Safety Factor =** | 1,270.88 | / | 265.69 | = | **4.78** |
|---|---|---|---|---|---|

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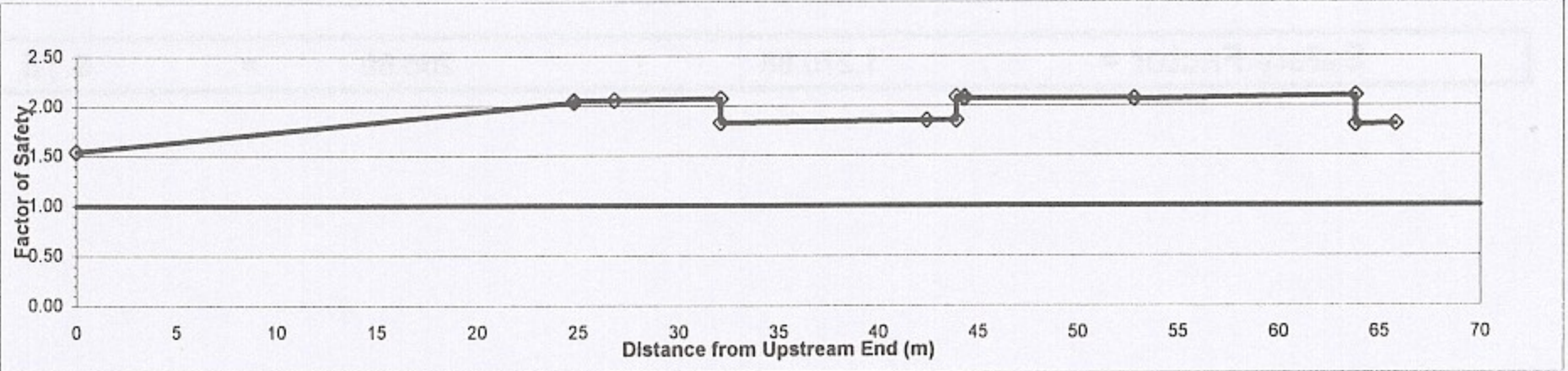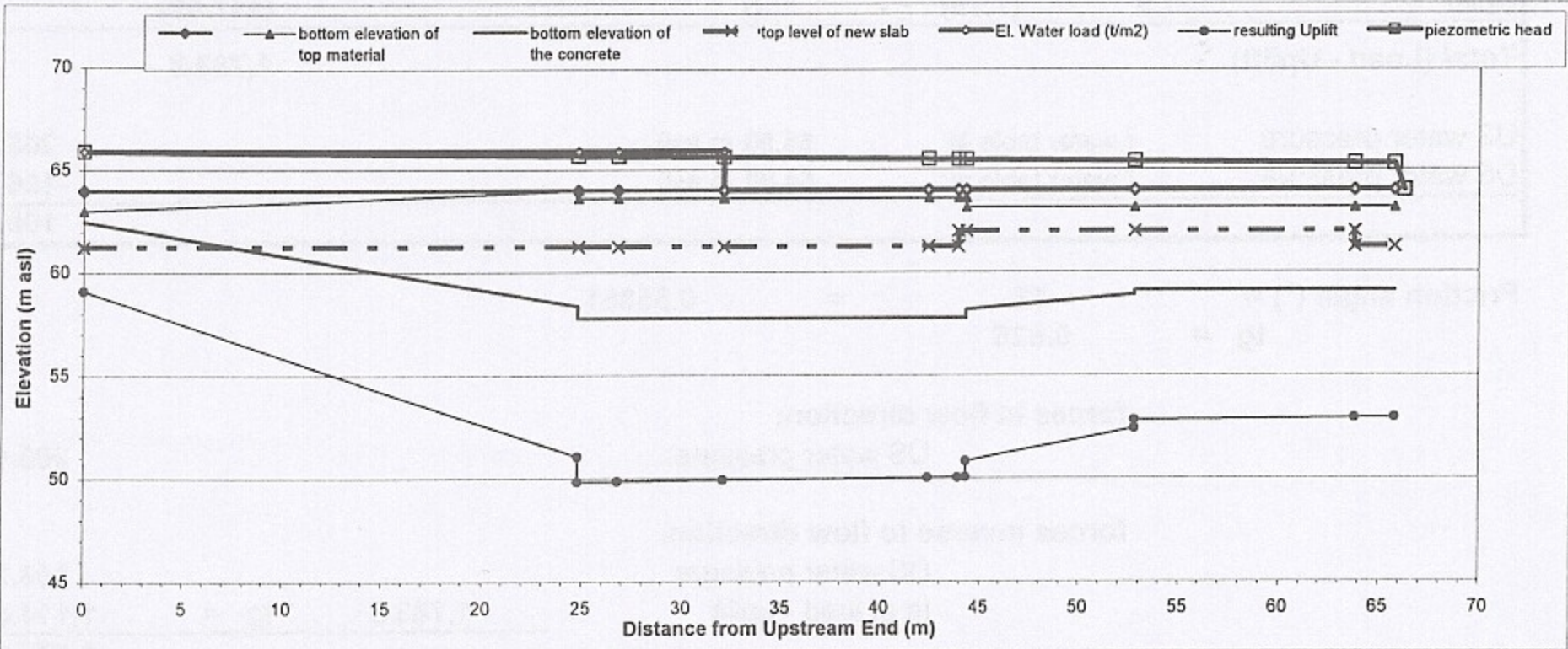September 2000

## Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case V:**     Normal - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 64.00 m as

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 16.235 | 8 | | | 688.4 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 16.135 | 8 | | | 245.3 | |
| Total load | 17.5 | | | | | 2,876.8 | |
| Uplift | 5.3 | (7.87) | 8 | | | (333.52) | |
| Uplift | 10.3 | (7.79) | 8 | | | (641.96) | |
| Uplift | 1.9 | (7.73) | 8 | | | (117.53) | |
| Total (Load - Uplift) | | | | | | 1,783.8 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 265.69 |
| add. water pressure (Westergaard) | | | | | | | 1.18 |
| horizontal force from dead load (0.1g) | | | | | | | 287.68 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 398.30 |

Friction angle (°) =     32     =     0.55851
       tg =     0.625

forces in flow direction:
       US water pressure:                                              554.55

forces inverse to flow direction:
       DS water pressure                                              156.25
       tg of load - uplift              1,783.8      * tg  =       1,114.63
                                                                    1,270.88

| **Safety Factor =** | 1,270.88 | / | 554.55 | = | **2.29** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Eastern Head Regulator

**Load Case V:**   Normal - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 64.00 m asl

| | |
|---|---|
| **Concrete Slab** | elevation: **63.00** m asl |

**Water level:**

Sand:
spec. weight:
**2.0**
t/m$^3$

upstream:   **65.90** m asl

**Sheet Pile Wall**

downstr.:   **64.00** m asl

vertical length at the end was considered with
**25** weights

elevation:
**58.75**
V

uplift force acting here =   (6.52) t/m$^2$

∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 8.50 t/m$^2$ |
| weight of add. water: | 1.00 t/m$^2$ |
| subtotal vertical forces | 9.50 t/m$^2$ |
| safety factor s =  vertical forces/uplift | 1.46 |
| Total vertical forces = | **2.98** t/m$^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 4.25 t/m$^2$ |
| remaining uplift from upstream: | (1.27) t/m$^2$ |
| subtotal | 2.98 t/m$^2$ |
| **safety factor** (weight of sand/remain. uplift) **s =** | **3.36** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 7.50 | =2+0.5+0.4+0.6+0.4+0.4+1.2+2 |
| horizontal distance along contact path Dh : | 65.80 | |
| total head loss H : | 1.90 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = **15.49** | > | **6 to 5** |
|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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**Load Case VI:  Unusual - New 75 cm slab,
headpond 65.90 m asl, tailwater 61.25 m asl**

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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case VI:**   Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 61.25 m asl

| | | |
|---|---|---|
| Water level US: | 65.90 | m asl |
| Water level DS: | 61.25 | m asl |
| D (H) = | 4.65 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage **128.30** m

...nce of DS Water Level to Bottom of Sheet Pile Wall **2.50** m

| | | US end of Pier | | Gate Section | | DS end of Pier | | | | | | Foundation Slab | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Distance from upstream end (m)** | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 128.30 | 103.50 | 103.50 | 101.50 | 101.50 | 98.20 | 98.20 | 85.90 | 85.90 | 84.40 | 84.40 | 84.00 | 84.00 | 75.50 | 75.50 | 64.50 | 64.50 | 62.50 | |
| top level of old slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 | #### |
| bottom elevation of top material | 60.25 | 61.25 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.85 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | 60.40 | |
| bottom elevation of the concrete | 59.75 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 2.90 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 3.10 | 2.25 | 1.50 | 1.30 | 1.30 | 1.30 | 1.30 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | | | |
| spec. weight (t/m³) | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 4.65 | 4.65 | 4.65 | 4.65 | 4.65 | 4.65 | - | - | - | - | - | - | - | - | - | - | - | - | - |
| **Total Weight (t/m²)** | 8.00 | 11.32 | 12.66 | 12.66 | 12.66 | 12.66 | 8.01 | 8.01 | 8.01 | 8.01 | 9.81 | 9.81 | 8.85 | 7.12 | 6.66 | 6.66 | 4.86 | 4.86 | |
| water l. under found. for stability analysis | 65.90 | 65.00 | 65.00 | 64.93 | 64.93 | 64.74 | 64.74 | 64.36 | 64.36 | 64.31 | 64.31 | 64.29 | 64.29 | 63.99 | 63.99 | 63.59 | 63.59 | 63.52 | #### |
| Uplift (t/m²) konst.p. | (1.50) | (2.90) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.50) | (3.10) | (2.35) | (2.15) | (2.15) | (2.15) | (2.15) | |
| Uplift (t/m²) linear p. | (4.65) | (3.75) | (3.75) | (3.68) | (3.68) | (3.49) | (3.49) | (3.11) | (3.11) | (3.06) | (3.06) | (3.04) | (3.04) | (2.74) | (2.74) | (2.34) | (2.34) | (2.27) | |
| **Resulting Uplift** | (6.15) | (6.65) | (7.25) | (7.18) | (7.18) | (6.99) | (6.99) | (6.61) | (6.61) | (6.56) | (6.56) | (6.54) | (6.14) | (5.09) | (4.89) | (4.49) | (4.49) | (4.42) | |
| **Total** | 1.85 | 4.67 | 5.41 | 5.48 | 5.48 | 5.67 | 1.02 | 1.40 | 1.40 | 1.45 | 3.25 | 3.27 | 2.70 | 2.03 | 1.77 | 2.17 | 0.37 | 0.44 | |
| **Safety Factor s =** | 1.30 | 1.70 | 1.75 | 1.76 | 1.76 | 1.81 | 1.15 | 1.21 | 1.21 | 1.22 | 1.50 | 1.50 | 1.44 | 1.40 | 1.36 | 1.48 | 1.08 | 1.10 | |

Chart legend: bottom elevation of top material — bottom elevation of the concrete — top level of new slab — El. Water load (t/m2) — resulting Uplift — piezometric head

Upper chart: Elevation (m asl) vs Distance from Upstream End (m)

Lower chart: Factor of Safety vs Distance from Upstream End (m)

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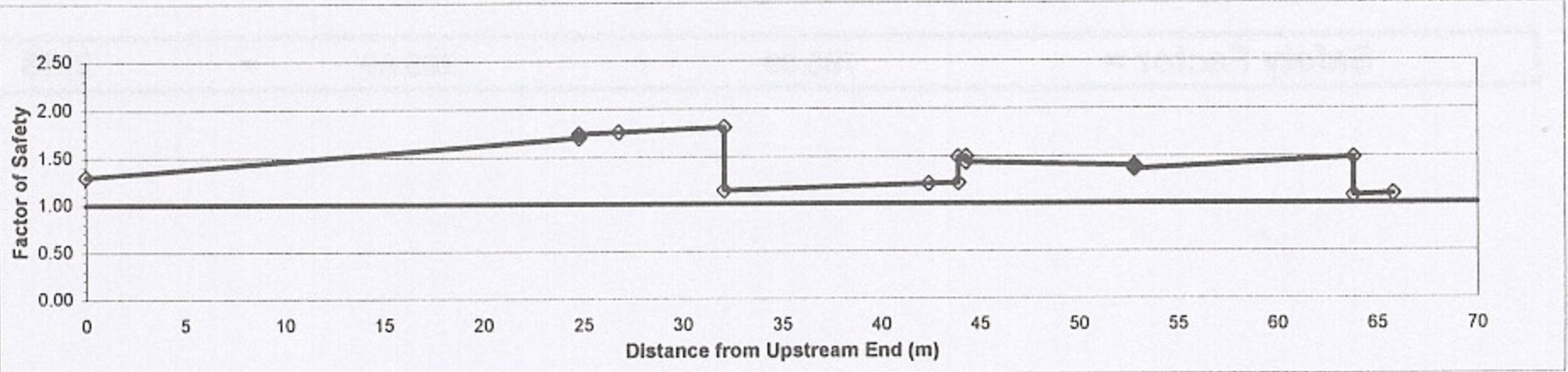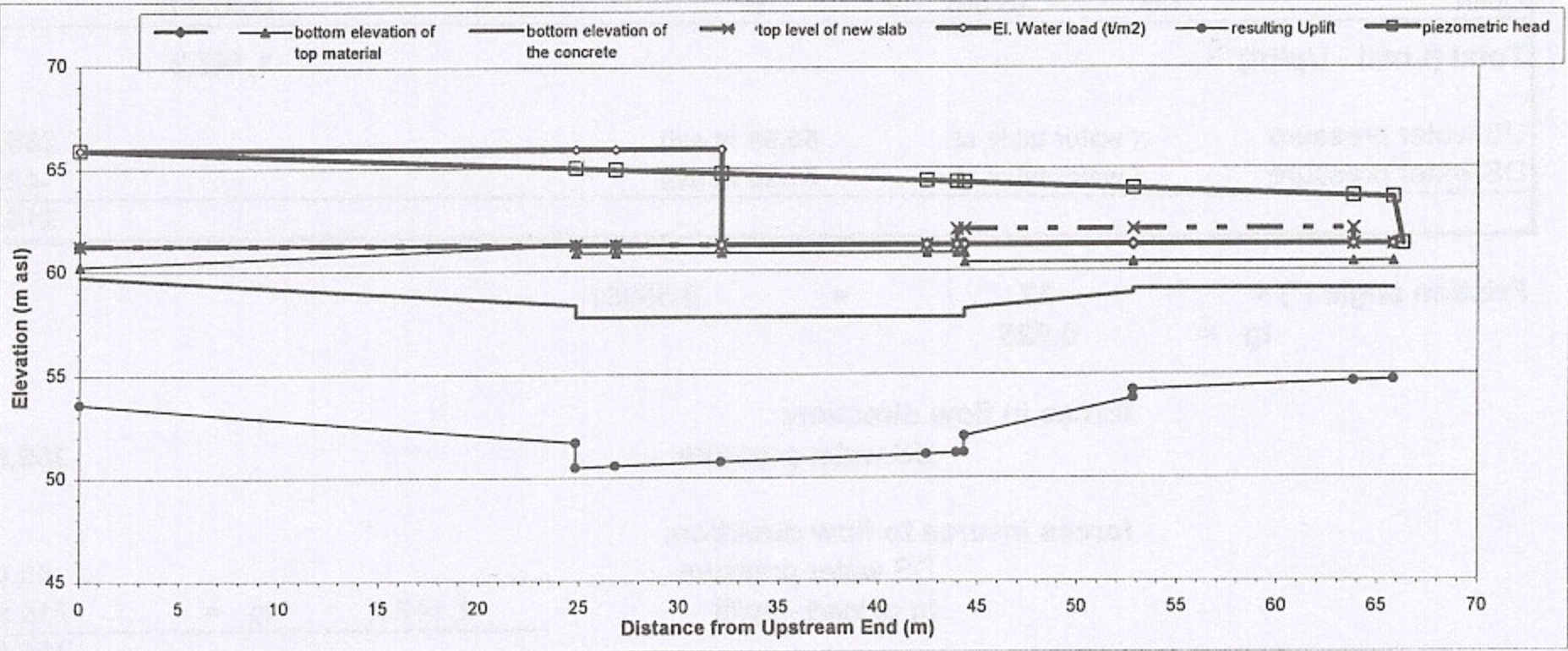
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## Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case VI:**   Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 61.25 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 12.66 | 8 | | | 536.8 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 9.81 | 8 | | | 149.1 | |
| Total load | 17.5 | | | | | 2,107.9 | |
| Uplift | 5.3 | (7.08) | 8 | | | (300.30) | |
| Uplift | 10.3 | (6.80) | 8 | | | (560.32) | |
| Uplift | 1.9 | (6.58) | 8 | | | (100.00) | |
| Total (Load - Uplift) | | | | | | 1,147.3 | |
| US water pressure | ( water table at: | | 65.90 m asl) | | | | 265.69 |
| DS water pressure | ( water table at: | | 61.25 m asl) | | | | -49.00 |
| | | | | | | | 216.69 |

Friction angle (°) =   32   =   0.55851
     tg   =   0.625

forces in flow direction:
    US water pressure:                                          265.69

forces inverse to flow direction:
    DS water pressure                                            49.00
    tg of load - uplift            1,147.3      * tg  =         716.89
                                                                765.89

| **Safety Factor =** | 765.89 | / | 265.69 | = | **2.88** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case VI:**   Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 61.25 m a

**Note:** from the foundation slab only the 4 m thick part is considered

|  | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 12.66 | 8 | | | 536.8 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 9.81 | 8 | | | 149.1 | |
| Total load | 17.5 | | | | | 2,107.9 | |
| Uplift | 5.3 | (7.08) | 8 | | | (300.30) | |
| Uplift | 10.3 | (6.80) | 8 | | | (560.32) | |
| Uplift | 1.9 | (6.58) | 8 | | | (100.00) | |
| Total (Load - Uplift) | | | | | | 1,147.3 | |

| | | | |
|---|---|---|---|
| US water pressure | ( water table at: | 65.90 m asl) | 265.69 |
| add. water pressure (Westergaard) | | | 7.06 |
| horizontal force from dead load (0.1g) | | | 210.79 |
| DS water pressure | ( water table at: | 61.25 m asl) | -49.00 |
| | | | 434.54 |

Friction angle (°) =      32      =      0.55851
              tg  =      0.625

forces in flow direction:
    US water pressure:                                              483.54

forces inverse to flow direction:
    DS water pressure                                               49.00
    tg of load - uplift              1,147.3      * tg  =      716.89
                                                                             765.89

| **Safety Factor =** | 765.89 | / | 483.54 | = | **1.58** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Eastern Head Regulator

**Load Case VI:**   Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 61.25 m asl

| | | |
|---|---|---|
| **Concrete Slab** | elevation: **63.00** m asl | **Water level:** |
| | Sand: spec. weight: | **upstream:** 65.90 m asl |
| **Sheet Pile Wall** | **2.0** t/m$^3$ | **downstr.:** 61.25 m asl |

vertical length at the end was considered with **25** weights

elevation: **58.75**

V

uplift force acting here = **(4.77)** t/m$^2$

∧ ∧ ∧ ∧ ∧ ∧ ∧

**Forces acting on bottom level of sheet pile wall:**

| | |
|---|---|
| weight of sand: | 8.50 t/m$^2$ |
| weight of add. water: | -1.75 t/m$^2$ |
| subtotal vertical forces | 6.75 t/m$^2$ |
| safety factor s =  vertical forces/uplift | 1.42 |
| Total vertical forces = | **1.98** t/m$^2$ |

**Considering all forces with uplift conditions:**

| | |
|---|---|
| weight of sand with uplift: | 4.25 t/m$^2$ |
| remaining uplift from upstream: | (2.27) t/m$^2$ |
| subtotal | 1.98 t/m$^2$ |

**safety factor** (weight of sand/remain. uplift) **s =**   **1.88**

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 7.50 | =2+0.5+0.4+0.6+0.4+0.4+1.2+2 |
| horizontal distance along contact path Dh : | 65.80 | |
| total head loss H : | 4.65 | |

**Weighted Creep Ratio WCR =**   ( Dv + Dh / 3 ) / H =   **6.33**   **>**   **6 to 5**

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :

| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |

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## Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case VI:**     Unusual - New 75 cm Slab, Headpond 65.90 m asl, Tailwater 61.25 m a

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 12.66 | 8 | | | 536.8 | |
| found. slab | 10.3 | 8.01 | 8 | | | 660.0 | |
| found. slab | 1.9 | 9.81 | 8 | | | 149.1 | |
| Total load | 17.5 | | | | | 2,107.9 | |
| Uplift | 5.3 | (7.08) | 8 | | | (300.30) | |
| Uplift | 10.3 | (6.80) | 8 | | | (560.32) | |
| Uplift | 1.9 | (6.58) | 8 | | | (100.00) | |
| Total (Load - Uplift) | | | | | | 1,147.3 | |
| US water pressure | ( water table at: | 65.90 m asl) | | | | | 265.69 |
| add. water pressure (Westergaard) | | | | | | | 7.06 |
| horizontal force from dead load (0.1g) | | | | | | | 210.79 |
| DS water pressure | ( water table at: | 61.25 m asl) | | | | | -49.00 |
| | | | | | | | 434.54 |

Friction angle (°) =        32          =        0.55851
        tg   =        0.625

forces in flow direction:
        US water pressure:                                                483.54

forces inverse to flow direction:
        DS water pressure                                                 49.00
        tg of load - uplift              1,147.3        * tg   =          716.89
                                                                         765.89

| **Safety Factor =** | 765.89 | / | 483.54 | = | **1.58** |
|---|---|---|---|---|---|

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# Load Case VII:  Extreme - New 75 cm slab,

# headpond 67.05 m asl, tailwater 64.00 m asl

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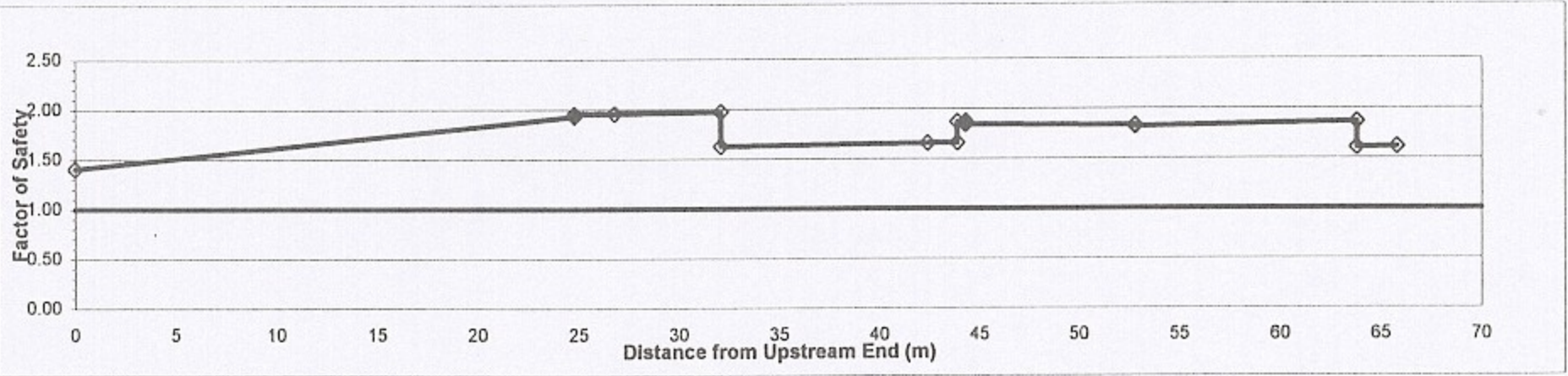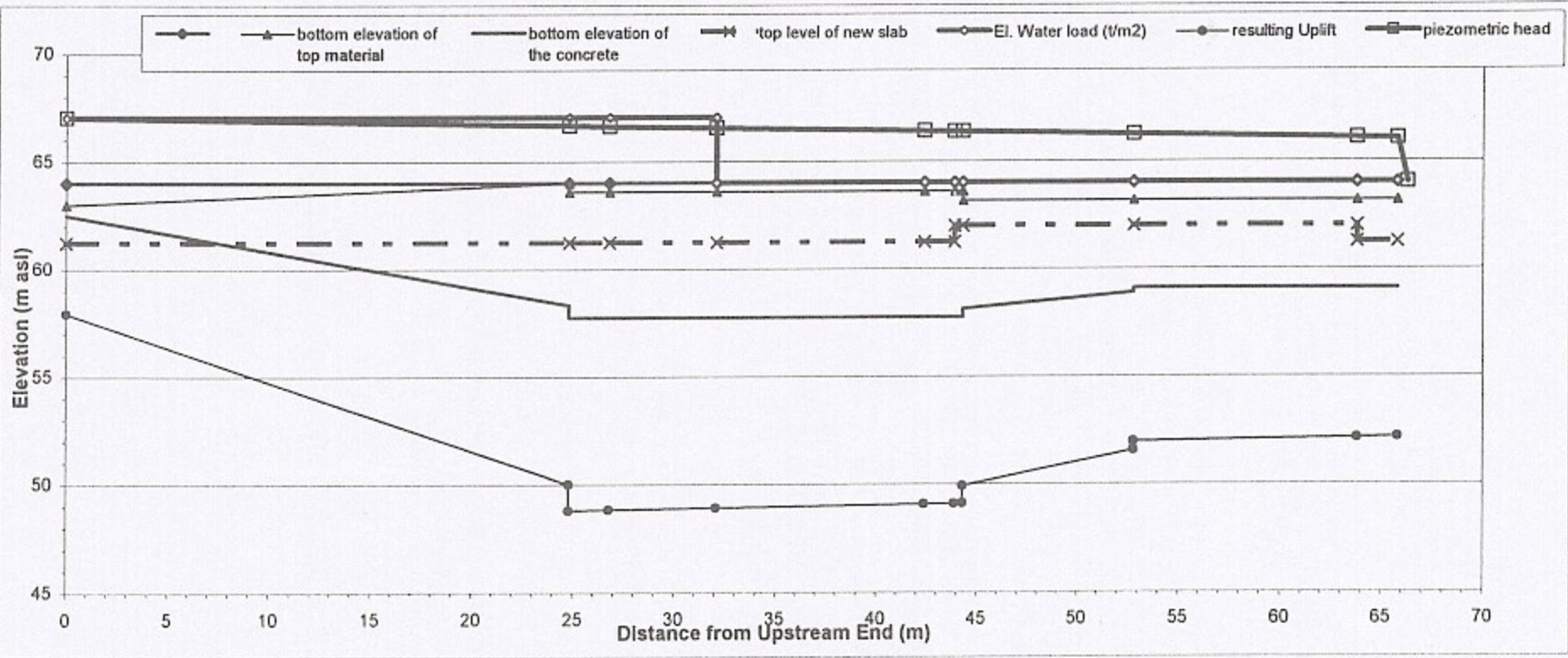Tender Design Report
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# Stability Analysis, Uplift
## Foundation Slab of the Eastern Head Regulator

**Load Case VII:**  Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m asl

| | | |
|---|---|---|
| Water level US: | 67.05 | m asl |
| Water level DS: | 64.00 | m asl |
| D (H) = | 3.05 | m |
| Length of slab: | 65.80 | m |
| Bottom of sheet pile wall at the end of slab: | 58.75 | m asl |

vertical length at the end was considered with **25** weights ==> length of drainage <u>197.05</u> m

nce of DS Water Level to Bottom of Sheet Pile Wall  5.25  m

| | | US end of Pier | Gate Section | | | | | | DS end of Pier | | | | Foundation Slab | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Distance from upstream end (m) | - | 24.80 | 24.80 | 26.80 | 26.80 | 32.10 | 32.10 | 42.40 | 42.40 | 43.90 | 43.90 | 44.30 | 44.30 | 52.80 | 52.80 | 63.80 | 63.80 | 65.80 | 66.3 |
| Chainage | 197.05 | 172.25 | 172.25 | 170.25 | 170.25 | 164.95 | 164.95 | 154.65 | 154.65 | 153.15 | 153.15 | 152.75 | 152.75 | 144.25 | 144.25 | 133.25 | 133.25 | 131.25 | |
| top level of old slab | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | 64.00 | |
| top level of foundation slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 | 61.25 |
| bottom elevation of top material | 63.00 | 64.00 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.60 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | 63.15 | |
| bottom elevation of the concrete | 62.50 | 58.35 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 57.75 | 58.15 | 58.90 | 59.10 | 59.10 | 59.10 | 59.10 | |
| top level of new slab | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 61.25 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 62.00 | 61.25 | 61.25 | |
| thickness of top material (m) | 1.00 | - | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.40 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | |
| spec. weight (t/m³) | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | 2.20 | |
| thickness of the concrete (m) | 0.50 | 5.65 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.85 | 5.00 | 4.25 | 4.05 | 4.05 | 4.05 | 4.05 | |
| spec. weight (t/m³) | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | 2.30 | |
| thickness of new concrete (m) | | | | | | | | | | | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | 0.75 | | | |
| spec. weight (t/m³) | | | | | | | | | | | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | 2.40 | | | |
| water load (t/m²) | 3.05 | 3.05 | 3.05 | 3.05 | 3.05 | 3.05 | - | - | - | - | - | - | - | - | - | - | - | - | |
| **Total Weight (t/m²)** | 6.40 | 16.05 | 17.39 | 17.39 | 17.39 | 17.39 | 14.34 | 14.34 | 14.34 | 14.34 | 16.14 | 16.14 | 15.17 | 13.46 | 12.99 | 12.99 | 11.19 | 11.19 | |
| water l. under found. for stability analysis | 67.05 | 66.67 | 66.67 | 66.64 | 66.64 | 66.55 | 66.55 | 66.39 | 66.39 | 66.37 | 66.37 | 66.36 | 66.36 | 66.23 | 66.23 | 66.06 | 66.06 | 66.03 | 64.00 |
| Uplift (t/m²) konst.p. | (1.50) | (5.65) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (6.25) | (5.85) | (5.10) | (4.90) | (4.90) | (4.90) | (4.90) | |
| Uplift (t/m²) linear p. | (3.05) | (2.67) | (2.67) | (2.64) | (2.64) | (2.55) | (2.55) | (2.39) | (2.39) | (2.37) | (2.37) | (2.36) | (2.36) | (2.23) | (2.23) | (2.06) | (2.06) | (2.03) | |
| **Resulting Uplift** | (4.55) | (8.32) | (8.92) | (8.89) | (8.89) | (8.80) | (8.80) | (8.64) | (8.64) | (8.62) | (8.62) | (8.61) | (8.21) | (7.33) | (7.13) | (6.96) | (6.96) | (6.93) | |
| **Total** | 1.85 | 7.73 | 8.47 | 8.50 | 8.50 | 8.58 | 5.53 | 5.69 | 5.69 | 5.71 | 7.51 | 7.52 | 6.96 | 6.11 | 5.85 | 6.02 | 4.22 | 4.25 | |
| **Safety Factor s =** | 1.41 | 1.93 | 1.95 | 1.96 | 1.96 | 1.97 | 1.63 | 1.66 | 1.66 | 1.66 | 1.87 | 1.87 | 1.85 | 1.83 | 1.82 | 1.86 | 1.61 | 1.61 | |

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# Stability Analysis, Sliding
## Foundation Slab of the Eastern Head Regulator

**Load Case VII:**     Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m a

Note: from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m³ | spec. weight t/m³ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 17.385 | 8 | | | 737.1 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 16.135 | 8 | | | 245.3 | |
| Total load | 17.5 | | | | | 2,925.5 | |
| Uplift | 5.3 | (8.84) | 8 | | | (374.99) | |
| Uplift | 10.3 | (8.72) | 8 | | | (718.81) | |
| Uplift | 1.9 | (8.63) | 8 | | | (131.16) | |
| Total (Load - Uplift) | | | | | | 1,700.6 | |
| US water pressure | ( water table at: | | 67.05 m asl) | | | | 345.96 |
| DS water pressure | ( water table at: | | 64.00 m asl) | | | | -156.25 |
| | | | | | | | 189.71 |

Friction angle (°) =          32          =          0.55851
       tg  =          0.625

forces in flow direction:
        US water pressure:                                          345.96

forces inverse to flow direction:
        DS water pressure                                           156.25
        tg of load - uplift              1,700.6     * tg =         1,062.64
                                                                   1,218.89

| **Safety Factor =** | 1,218.89 | / | 345.96 | = | **3.52** |
|---|---|---|---|---|---|

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# Stability Analysis, Sliding with Earthquake of 0.1 g
## Foundation Slab of the Eastern Head Regulator

**Load Case VII:**     Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m a

**Note:** from the foundation slab only the 4 m thick part is considered

| | length (in flow dir.) | height m | depth m | volume m$^3$ | spec. weight t/m$^3$ | weight t | hor. forces t |
|---|---|---|---|---|---|---|---|
| wall A | 1.15 | 2.50 | 6.00 | 17.3 | 2.2 | 38.0 | |
| wall B | 0.75 | 4.80 | 6.00 | 21.6 | 2.2 | 47.5 | |
| wall C | 0.80 | 2.50 | 6.00 | 12.0 | 2.2 | 26.4 | |
| road | 6.00 | 1.40 | 6.00 | 50.4 | 2.2 | 110.9 | |
| pier top | 10.00 | 1.70 | 2.00 | 34.0 | 2.2 | 74.8 | |
| pier medium | 12.78 | 4.20 | 1.90 | 102.0 | 2.2 | 224.4 | |
| pier bottom | 14.18 | 4.05 | 1.90 | 109.1 | 2.2 | 240.1 | |
| Total Pier | | | | 346.3 | | 762.0 | |
| | | height x spec.w. | | | | | |
| found. slab | 5.3 | 17.385 | 8 | | | 737.1 | |
| found. slab | 10.3 | 14.335 | 8 | | | 1,181.2 | |
| found. slab | 1.9 | 16.135 | 8 | | | 245.3 | |
| **Total load** | **17.5** | | | | | **2,925.5** | |
| Uplift | 5.3 | (8.84) | 8 | | | (374.99) | |
| Uplift | 10.3 | (8.72) | 8 | | | (718.81) | |
| Uplift | 1.9 | (8.63) | 8 | | | (131.16) | |
| **Total (Load - Uplift)** | | | | | | **1,700.6** | |

| | | | | |
|---|---|---|---|---|
| US water pressure | ( water table at: | 67.05 m asl) | | 345.96 |
| add. water pressure (Westergaard) | | | | 3.04 |
| horizontal force from dead load (0.1g) | | | | 292.55 |
| DS water pressure | ( water table at: | 64.00 m asl) | | -156.25 |
| | | | | 485.30 |

| | | | | |
|---|---|---|---|---|
| Friction angle (°) = | 32 | = | 0.55851 | |
| tg  = | 0.625 | | | |

forces in flow direction:
US water pressure:     641.55

forces inverse to flow direction:
DS water pressure     156.25
tg of load - uplift    1,700.6   * tg  =   1,062.64
    1,218.89

| **Safety Factor =** | 1,218.89 | / | 641.55 | = | **1.90** |
|---|---|---|---|---|---|

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## Stability Analysis, Sand at the End of Slab
## Foundation Slab of the Eastern Head Regulator

**Load Case VII:**  Extreme - New 75 cm Slab, Headpond 67.05 m asl, Tailwater 64.00 m asl

67.05

| | | | |
|---|---|---|---|
| **Concrete Slab** | elevation: **63.00** m asl | **Water level:** | |
| | Sand: spec. weight: **2.0** t/m$^3$ | upstream: | 67.05 m asl |
| **Sheet Pile Wall** | | downstr.: | 64.00 m asl |

elevation: **58.75**

V

vertical length at the end was considered with **25** weights

uplift force acting here =          (7.28) t/m$^2$

^ ^ ^ ^ ^ ^ ^

| Forces acting on bottom level of sheet pile wall: | |
|---|---|
| weight of sand: | 8.50 t/m$^2$ |
| weight of add. water: | 1.00 t/m$^2$ |
| subtotal vertical forces | 9.50 t/m$^2$ |
| safety factor s = vertical forces/uplift | 1.30 |
| Total vertical forces = | **2.22** t/m$^2$ |

| Considering all forces with uplift conditions: | |
|---|---|
| weight of sand with uplift: | 4.25 t/m$^2$ |
| remaining uplift from upstream: | (2.03) t/m$^2$ |
| subtotal | 2.22 t/m$^2$ |
| **safety factor** (weight of sand/remain. uplift) **s =** | **2.09** |

## Stability Analysis, Piping according to Lane:

(see: Geotechnical Analysis - I.Dunn, L.Anderson, F.Kiefer - John Wiley & Sons - 1980)

| | | |
|---|---|---|
| vertical distance along contact path Dv : | 7.50 | =2+0.5+0.4+0.6+0.4+0.4+1.2+2 |
| horizontal distance along contact path Dh : | 65.80 | |
| total head loss H : | 3.05 | |

| **Weighted Creep Ratio WCR =** | ( Dv + Dh / 3 ) / H = | **9.65** | > | **6 to 5** |
|---|---|---|---|---|

The dam is considered **safe** with respect to erosion if WCR > WCRcr

Critical weight creep ratios WCRcr :
| | |
|---|---|
| - Very fine sand or silt to fine sand | 8.5 to 7 |
| - **Medium to coarse sand** | **6 to 5** |
| - Fine to coarse gravel | 4 to 3 |
| - Boulders with some cobbles and gravel | 2.5 |
| - Soft to medium clay | 3 to 2 |
| - Hard to very hard clay | 1.8 to 1.6 |